

Stormwater Drainage Manual

Research Engineers
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Acknowledgements

The original HERPICC "County Storm Drainage Manual" was written by Dr. Christopher Burke in 1981. Several revisions have taken place over the last 14 years. The latest revision, now titled HERPICC "Stormwater Drainage Manual" encompasses the basis and theory that constitutes stormwater drainage contained in the original manual and expands many areas where current information or methods have made available new data and techniques. For example, a study performed at Purdue University developed new Intensity-Duration-Frequency (I-D-F) curves for several cities throughout Indiana by using hourly rainfall data from July 1948 to June 1991 (Purdue, et al., 1992). These curves are presented for determining rainfall data for the various cities in the state where the data was analyzed. Chen's method, is then presented for determining rainfall values in the areas of the state where I-D-F curves have not been developed. Another example of encompassing current techniques is the addition of Chapter 8, which focuses on two computer programs that are widely used for stormwater drainage computations, and the references throughout the manual to various programs available for simplifying detailed hand calculations.

In the process of completely revising the manual, there have been several people who have contributed a great deal of their time and efforts. David McCormick, graduate student, School of Civil Engineering, has worked in detail on helping update Chapters 2 and 3 by introducing some of the methods used for determining rainfall values and the quantities of runoff. Dr. A. R. Rao, Professor of Environmental and Hydraulics Engineering, School of Civil Engineering, has assisted throughout the revision by reviewing and suggesting new methods that were implemented in the manual. Ms. Lynn Stocksick, of the HERPICC staff, has aided in the typing of many chapters and appendices. For all of their time and efforts we are most appreciative.

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HERPICC

STORMWATER DRAINAGE MANUAL

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Chapter 1

INTRODUCTION

Chapter 1 - INTRODUCTION

Indiana, like many parts of the country, is experiencing a rapid change in the character of its land use. Areas which were once predominantly rural are now being developed for urban or suburban use. The consequence of covering once pervious soils with concrete, asphalt and buildings is a decrease in the rainfall quantity which may infiltrate and a subsequent increase in the runoff volume. In addition, components of the drainage system such as sewers, gutters and streets, convey this increased volume to the point of disposal much quicker than in the rural condition. The result of this "urbanization" is a significant increase in the volume of stormwater and a conveyance rate higher than that experienced in the undeveloped rural state.

Engineers, surveyors or others involved with storm drainage design are faced with the task of designing drainage systems that are economical and at the same time provide adequate protection to minimize the loss of property or life. This manual has been compiled to provide the designer with resource materials which will help in meeting this challenge.

The information presented in this manual is not necessarily original or unique. It is a comprehensive catalog of procedures, design methods and criteria, and general background information which will enable the designer to quickly learn or review the basic principles and applications of storm drainage design. This information is currently dispersed in many other texts and manuals and is not readily available as a single source.

The manual presents eight chapters along with four appendices. Each chapter presents an introduction and background information about the subject(s) discussed. Following the introduction is a presentation of the appropriate equations, graphs, charts or tables for the methods which are employed in drainage design. Each chapter includes example problems which illustrate the application of the material presented. References at the end of the chapter provide the reader with additional sources of information.

Chapter 2 presents the precipitation and hydrologic cycle which is the starting point of any drainage design. The processes involved in the formation of rainfall are presented, along with a discussion of the temporal and areal distribution of rainfall. A discussion of the collection and analysis of precipitation follows, along with a statistical analysis and hydraulic risk. Depth and intensity-duration-frequency equations for several cities throughout Indiana are given and intensity-duration-frequency curves for the entire State of Indiana are presented. Chen's method is discussed to determine rainfall depths where intensity-duration-frequency equations have not been developed. The chapter includes example problems illustrating the application of the Huff curves to generate a time distribution of rainfall, use of the statistical analysis and hydraulic risk associated with rainfall, determination of a rainfall intensity using an intensity-duration-curve and Chen's method. The final example compares the rainfall intensities obtained from the intensity-duration-frequency curves and those from Chen's method.

Chapter 3 presents the phenomenon of runoff and its estimation, which is the most important aspect of drainage design. The various components which affect runoff are presented along with six methods for estimating the amount of runoff from a rainfall event. The first method is the popular rational method which computes a peak runoff rate only. The second procedure outlined is the Soil Conservation Curve Number Method which computes a volume of runoff. The third procedure outlined is the use of hydrographs. This includes unit hydrographs, dimensionless unit hydrographs and storm hydrographs. The fourth method provided is the Water Resource Council Method which evaluates a series of discharge data to obtain the flowrate corresponding to a desired period. Statistical analysis of peak discharges, which is very similar to the techniques used in analyzing rainfall data, is the fifth method. The last procedure is the coordinated discharges method for selected streams in Indiana. Example problems illustrate applications of all of these procedures.

Since open channels are the primary conveyances employed in storm drainage design, they are discussed in Chapter 4. The chapter presents a discussion of channel geometry, flow classification and applications of the energy equation. Next, the appropriate equations for computing uniform flow, specific energy, critical flow and flow in a floodplain are presented. Design criteria used in the selection of location, channel cross-section, roughness coefficients and lining are then given. The text portion of the chapter concludes with the analysis of gradually-varied flow and its application to backwater curves. The example problems illustrate most of the methods presented. A brief introduction to the computer program HY-8 is given and an example is provided.

Regardless of a drainage systems' capacity, it must have inlets which will allow the stormwater into the system. Chapter 5 discusses the methods used for sizing inlets and gutters. The chapter begins with a discussion of flow in gutters and methods used in properly estimating gutter capacity. The estimation of inlet capacity for gutter, curb, slotted drain and combined inlets for continuous grades and sump conditions is presented. The text portion of the chapter concludes with design criteria for inlet design, including inlet spacing using the rational method. Example problems illustrate methods used in computing flow in gutters, gutter inlets and curb inlets for both a continuous grade and sump condition and a slotted drain inlet for sump condition. The last example problem illustrates the spacing of a gutter inlet using the rational method.

One important element of drainage design is stormwater storage; this topic is presented in Chapter 6. The chapter starts with a discussion of all the types of storage facilities which may be employed and follows with a discussion of two methods (outlined in Chapter 3) which can be used to compute the volume of storage needed: the rational method and the SCS Hydrograph Method. The text portion of the chapter designates the criteria used in designing retention and detention ponds, and parking lot, rooftop and infiltration facilities. A discussion of devices used for regulating outflow is also presented. Example problems present applications of the rational method, curve number method and hydrograph method. The fourth problem illustrates methods for sizing a multi-component facility site. Lastly, the SCS TR-55 methodology for volume calculation is presented.

The design of a storm sewer system is presented in Chapter 7, using the methods and procedures presented in all the previous chapters (2-6). A general introduction is followed by a discussion of the methods used in the sizing of storm sewers, including the rational method and a computer program method. A brief introduction to the hydraulics of culverts is presented, along with design criteria for designing storm sewer systems. The chapter concludes with a presentation of the types of pipe material which may be used for storm sewers. Example problems illustrate the application of the rational method to hypothetical drainage basin and to an actual subdivision.

Finally, Chapter 8 presents two computer applications for computing watershed runoff. This chapter incorporates the concepts and procedures in Chapter 2 and 3 and simplifies the calculations by using the computer programs TR-20 and HEC-1. A description of the program is followed by three example problems. The same example problems are used for both applications. The TR-55 and t_c methods that are affiliated in the TR-20 program have some forms and worksheets that are included in Appendix D.

The manual concludes with four appendices. Appendices A and B present background material for statistical analysis and the fundamentals of hydraulics. Appendix C outlines regulatory agencies and governmental bodies which may have jurisdiction over drainage projects. Appendix D contains blank TR-20 forms.

The basics of statistical analysis included in Appendix A, consists of the general concepts of the mean, standard deviation and probability. The Gumbel and Log-Pearson Type III distributions are presented, along with an example showing the various aspects of the material presented. Also, rainfall depth curves for the continental United States are provided.

The fundamentals of hydraulics in Appendix B presents a general review of hydraulic principles needed by the drainage engineer. This includes the law of conservation of mass, continuity equation, and the concepts of pressure and energy. A discussion of pipes flowing under pressure is presented, along with the Darcy-Weisbach and Hazen-Williams equations and a discussion of minor losses and flow in series and parallel pipe networks. A summary of some of the elements of open channel flow (Chapter 4) concludes Appendix B.

Appendix C contains a list of regulatory agencies which may have jurisdiction over drainage projects. A discussion of the local, state, and Federal organizations from which the designer may need to get approval is presented, along with citations to applicable statutes and regulations.

A separate manual is now available to provide documentation for the HERPICC Stormwater Drainage Manual disk. This disk provides spreadsheets for many of the example problems that were calculated using spreadsheets in Chapters 2 through 7. The disk can be obtained through the HERPICC office and is available in Quattro Pro for DOS, Quattro Pro for Windows, and Excel for Windows formats

Chapter 2

RAINFALL

Chapter 2 - RAINFALL

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Chapter 2 - RAINFALL

LIST OF PARAMETERS

a_1, d', n'	Coefficients for determining rainfall depth by Chen's method
c, d, α, β	Regional coefficients for determining rainfall intensity-duration-frequency curves
C_s	Skewness coefficient
ΔD	Increment of depth (inches)
i	Rainfall intensity
J	Risk
K	Gumbel frequency factor
K_t	Log Pearson Type III frequency factor
m	Ranking of event
n	Total number of events or years
p	Rainfall depth
\bar{p}	Arithmetic average of rainfall data
p_i	Individual extreme rainfall value
P	Frequency of an event being equalled or exceeded
s	Standard deviation
s_y	Standard deviation of y
t	Storm duration (hours)
Δt	Increment of time (hours)
T_r	Return Period
X	p_1^{100} / p_1^{10}
\bar{y}	Arithmetic average of the logarithm of rainfall data
y_i	Logarithm of individual extreme rainfall value

Chapter 2 - RAINFALL

2 1 - INTRODUCTION

In the design of drainage systems, rainfall provides the input to deterministic methods or models. This input may be from statistical analysis of data or from an actual rainfall event. The basic aspects of the hydrologic cycle, of rainfall data and the precipitation process, and how precipitation varies in space and time, as well as a discussion of the collection of rainfall data and the statistical analysis of this data, are included.

2.1 1 - Hydrologic Cycle

Precipitation is a part of the hydrologic cycle. The hydrologic system is illustrated in Figure 2 1 1.

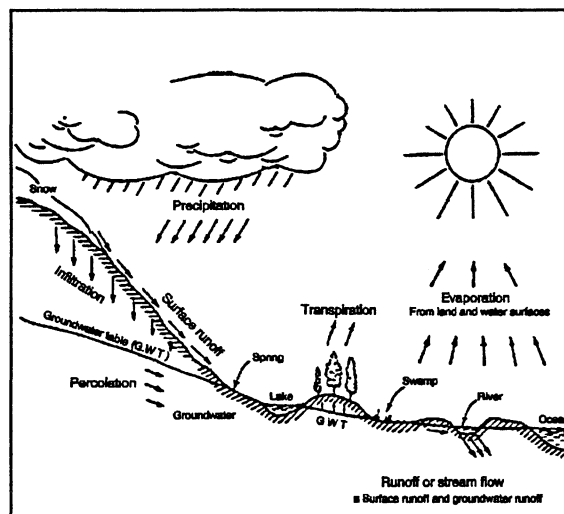


Figure 2 1 1 The Water Cycle (Fair et al , 1971)

2 1 2 - Precipitation Processes

While water vapor is a necessary factor in the formation of precipitation, it is not the sole requirement. Three basic steps are necessary for precipitation to form. First, a saturated condition in the atmosphere must exist. This condition is brought about mostly through the cooling which accompanies an ascending body of moist air. The saturated condition

the transformation of water in the air mass from vapor to liquid or solid state. This transformation, which is called condensation, occurs on small hygroscopic particles called condensation nuclei. As the air mass is lifted, expanded and cooled, the water vapor will condense on these particles to initiate the formation of precipitation. Snow will follow a similar process. Growth of the small water droplets to larger "precipitable" size is the third step. As the droplets form, there are influences which reduce their size through evaporation. Solar energy is one such influence. Heat generated from the physical transformation of the state of water molecules coupled with air movement, also contributes to evaporation. If evaporation is too great, then rainfall does not occur.

2.1.3 - Time Distribution of Rainfall

Time distribution of rainfall is important in the planning, sizing, and design of urban stormwater management systems. A complete approach to describing the time distribution of rainfall would include its probabilistic nature. One such study was performed in which data were collected over a 400 square mile area in east-central Illinois utilizing 40 rain gages. (Huff, 1970; Viessman, 1977).

The study by Huff (1970) found that the major portion of the total storm rainfall occurs in a small part of the total storm, regardless of storm duration, areal mean rainfall, and total number of showers or bursts in the storm period. The storms were classified into four groups (1st, 2nd, 3rd, and 4th quartiles) depending on the quartile, defined as a 25% time segment of the total storm duration, in which the greatest amount of total rainfall occurred (Huff, 1970; 1972). Using the Huff methodology, Tables 2.1.1 - 2.1.9 were derived from a study of rainfall at four stations (Indianapolis, Evansville, Fort Wayne, and South Bend) in the State of Indiana (Purdue et al., 1992). The information for Indianapolis is shown graphically in Figure 2.1.2. The axes of the curves are dimensionless cumulative rainfall and cumulative storm time. Each curve represents a different probability level. For example, a 10% probability curve may be interpreted as the distribution of rainfall that was exceeded in 10% of the storms. Example 2.1.1 illustrates the use of this data.

The Huff quartile groups represent typical rainfall distributions for 4 different storm duration ranges. Generally, in water resources modeling, the first quartile is taken to apply to storms less than or equal to 6 hours in duration. The second quartile is for storms greater than 6 hours and less than or equal to 12 hours while the third Huff quartile is for storms greater than 12 hours and less than or equal to 24 hours. Fourth quartile storms apply to storm durations greater than 24 hours (IDOT DWR, 1992).

Table 2.1.1
10% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	42.00	16.36	14.36	19.35	45.44	15.56	13.64	17.14	42.68	15.00	13.24	17.50	39.65	16.67	15.38	17.50
20	64.35	32.73	25.26	30.00	69.63	32.50	23.62	28.75	63.64	31.43	24.57	28.75	63.67	30.92	25.00	28.00
30	76.36	58.10	33.33	36.36	79.14	60.00	30.83	37.50	78.18	55.31	33.45	37.60	75.58	54.91	32.76	38.69
40	83.84	76.36	41.82	43.53	86.09	77.27	40.00	43.40	85.26	75.00	41.67	42.86	82.52	73.33	41.20	45.00
50	89.63	87.50	54.72	50.00	89.09	88.72	53.85	50.00	89.05	86.70	54.55	50.00	87.68	85.71	53.45	50.00
60	92.50	92.54	78.18	54.55	92.85	93.94	77.62	55.17	92.00	91.30	77.10	55.56	90.91	91.30	77.30	57.14
70	95.00	95.69	92.35	63.64	95.00	95.77	90.33	62.55	94.40	95.12	91.20	63.64	93.68	94.13	90.89	63.60
80	97.00	97.00	96.43	82.50	96.97	97.50	96.30	83.33	96.51	97.14	96.67	82.82	96.15	96.56	95.29	81.82
90	98.57	98.67	98.73	96.73	98.78	98.98	98.78	96.92	98.18	98.57	98.57	96.84	98.21	98.35	98.16	96.09
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.2
20% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	31.71	12.17	10.71	14.29	36.67	12.28	10.00	13.23	32.73	11.67	10.59	14.29	31.93	12.73	12.35	14.29
20	52.92	27.27	21.09	25.00	60.00	27.27	18.67	23.28	57.58	25.71	20.00	25.00	53.33	26.67	20.86	25.00
30	68.00	50.00	28.57	31.79	71.15	49.40	26.86	30.41	70.00	47.06	28.46	31.92	68.00	47.27	28.44	33.33
40	76.67	67.78	35.83	38.00	78.57	69.38	35.19	37.61	77.78	66.25	36.60	38.00	76.00	65.19	36.88	40.00
50	82.93	80.77	50.00	44.44	85.19	82.31	48.08	44.12	82.73	79.41	49.06	43.90	80.77	79.41	48.00	46.15
60	88.57	88.89	72.50	50.00	88.57	89.38	70.38	51.43	87.44	87.50	69.55	50.00	85.71	86.38	69.23	51.28
70	92.20	92.31	88.89	60.32	92.35	93.29	87.82	60.00	90.63	92.45	85.71	59.00	90.00	91.38	85.25	59.81
80	95.12	95.43	94.57	77.33	95.23	96.12	94.49	79.05	94.29	95.51	93.77	77.78	93.75	94.44	92.71	77.14
90	97.94	97.85	98.14	94.60	97.96	98.40	98.09	95.00	97.27	97.78	97.86	94.49	97.00	97.49	97.10	93.55
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.3
30% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	26.67	9.88	8.47	10.00	30.00	9.86	8.00	10.00	26.92	9.47	8.57	11.61	26.67	10.61	10.00	12.00
20	48.57	23.81	16.92	20.00	52.80	22.98	16.36	20.00	50.00	22.86	16.67	20.16	47.69	24.00	18.57	21.22
30	62.86	44.74	24.00	28.00	66.67	42.77	24.00	26.79	63.33	42.00	23.59	27.59	62.00	41.82	25.00	28.57
40	71.22	62.91	31.79	33.33	74.19	62.50	32.00	33.67	71.05	60.32	33.15	33.33	70.00	60.00	33.33	35.00
50	77.42	76.35	45.61	40.63	80.00	77.14	44.44	40.44	77.78	75.13	45.16	40.00	76.25	74.19	44.95	41.67
60	83.81	83.87	66.47	47.69	85.45	85.00	67.34	47.65	83.33	84.05	64.29	46.67	81.82	82.11	64.71	49.13
70	89.76	90.00	84.73	56.94	89.39	90.73	84.21	57.14	87.60	89.76	82.57	55.71	87.60	87.80	82.00	56.52
80	93.50	94.03	92.50	74.62	93.55	94.73	92.50	75.83	92.22	93.62	91.43	73.85	91.20	92.73	90.70	74.17
90	97.22	97.20	97.36	92.51	97.33	97.88	97.30	93.16	96.25	97.09	96.94	92.22	95.71	96.71	96.11	91.20
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.4
40% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	23.29	7.74	6.42	8.42	27.00	8.00	6.45	8.00	23.33	7.79	7.00	9.19	22.92	8.85	8.33	10.00
20	44.62	20.87	14.00	16.96	48.48	20.00	13.08	16.64	45.40	20.00	14.29	17.30	42.86	21.05	15.76	18.73
30	58.95	40.00	20.62	24.79	60.74	37.20	20.29	23.75	58.33	37.97	21.05	23.81	56.67	37.05	22.00	25.00
40	66.67	57.78	28.57	29.84	69.83	56.93	28.81	30.00	66.67	55.71	29.69	30.00	65.36	55.65	30.00	32.31
50	73.53	72.22	41.94	37.21	75.71	73.68	40.91	37.66	72.41	71.43	42.11	35.71	71.43	70.25	41.82	38.89
60	80.24	80.28	61.97	44.53	81.22	81.82	61.12	43.61	78.57	80.00	60.83	42.55	77.89	78.95	62.04	44.44
70	86.15	87.14	81.60	54.00	87.12	88.13	80.17	54.12	85.00	87.44	79.05	52.50	84.00	85.60	78.67	53.33
80	91.30	92.08	90.77	71.69	92.17	93.18	90.63	72.73	90.00	91.72	89.82	70.94	89.23	90.96	88.67	70.77
90	96.15	96.55	96.50	90.00	96.58	97.14	96.58	91.43	95.00	96.25	96.00	90.00	94.55	95.88	95.33	89.22
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.5
50% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	20.00	6.50	5.26	6.67	22.82	6.28	5.13	6.92	20.00	6.67	6.00	7.14	20.00	7.50	7.00	8.26
20	40.80	18.13	11.55	14.25	44.69	17.33	11.11	14.04	41.11	17.14	12.23	14.23	40.00	18.57	13.33	16.35
30	54.95	35.85	17.06	20.00	57.11	33.33	16.67	20.51	54.83	34.17	18.86	20.00	51.67	34.00	20.00	22.73
40	62.50	52.94	24.24	26.09	65.33	53.09	25.44	27.06	62.00	52.18	26.15	25.71	60.89	51.43	27.50	28.50
50	68.75	67.86	37.78	33.33	71.43	69.57	37.93	34.21	68.42	66.67	38.46	33.33	67.35	66.67	39.13	34.04
60	76.67	76.52	58.33	40.00	78.15	78.57	57.39	40.91	75.00	76.36	57.23	38.00	75.00	75.17	58.46	40.20
70	83.05	83.81	78.03	50.00	84.66	85.60	77.44	50.79	81.62	84.29	76.11	48.50	80.83	82.32	75.98	50.00
80	89.70	90.67	88.68	68.57	90.00	91.72	88.54	69.70	87.50	90.00	87.69	68.24	86.67	88.89	86.79	67.50
90	95.00	95.89	95.29	88.37	95.36	96.50	95.88	89.36	93.75	95.56	95.08	87.88	92.89	94.78	94.17	87.50
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.6
60% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	17.50	5.22	4.18	5.45	19.15	5.07	4.00	5.22	17.78	5.71	4.67	5.45	17.37	6.09	5.65	6.67
20	37.86	15.56	9.23	11.03	40.51	15.12	9.00	11.11	38.00	14.69	10.42	11.00	36.00	15.94	11.25	13.33
30	50.00	31.25	14.29	17.39	53.13	30.00	13.69	16.94	50.00	30.00	15.89	16.04	48.57	30.53	17.14	20.00
40	58.33	48.39	21.58	22.28	60.86	48.81	22.22	23.00	57.50	48.42	22.44	22.98	56.67	48.00	24.00	25.00
50	65.12	64.13	32.45	28.57	66.95	65.79	33.96	30.77	65.00	63.64	35.00	27.42	65.00	62.50	35.66	29.41
60	71.97	73.33	54.17	36.67	74.36	75.56	53.33	37.23	71.43	73.33	53.08	34.17	71.43	72.44	54.07	37.02
70	80.00	81.13	73.89	47.14	80.91	82.46	74.43	47.27	77.78	81.43	72.86	44.17	77.78	79.29	72.67	47.02
80	86.67	88.82	86.32	65.00	87.50	89.53	86.67	66.00	85.00	88.00	85.71	64.41	84.00	86.67	84.44	64.44
90	93.33	94.74	94.12	86.67	94.00	95.50	94.67	85.00	92.12	94.44	93.85	85.42	90.77	93.55	92.89	85.71
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.7
70% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	14.44	4.12	3.16	4.00	15.88	4.08	3.13	3.75	15.00	4.62	3.64	4.13	14.29	5.00	4.38	4.62
20	35.00	13.19	7.14	8.26	36.67	12.33	7.10	8.72	35.00	12.73	8.42	8.33	33.33	13.68	10.00	10.53
30	47.01	26.36	11.76	14.21	49.88	25.88	11.33	14.00	46.67	26.00	13.04	12.86	45.00	26.79	14.89	14.59
40	53.65	44.21	18.10	18.46	56.79	44.40	17.55	18.18	53.87	44.08	18.82	17.24	52.86	44.24	20.41	20.55
50	60.71	60.81	28.91	24.88	63.37	61.11	29.30	25.00	61.29	60.00	30.77	24.34	60.71	59.46	31.37	25.00
60	68.00	70.59	48.89	31.43	70.00	71.58	49.41	32.86	67.50	70.00	49.52	30.26	67.65	68.40	50.48	31.58
70	75.00	77.84	70.65	42.50	76.54	79.76	70.34	43.33	75.00	78.00	70.00	40.00	75.00	76.47	70.00	41.74
80	83.38	86.00	84.44	61.05	85.00	87.32	84.08	61.33	82.35	85.71	83.28	60.00	80.00	83.89	82.19	60.00
90	91.25	93.44	93.00	84.54	92.50	94.11	93.62	82.04	90.50	93.16	92.73	82.50	88.75	91.82	91.25	82.50
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

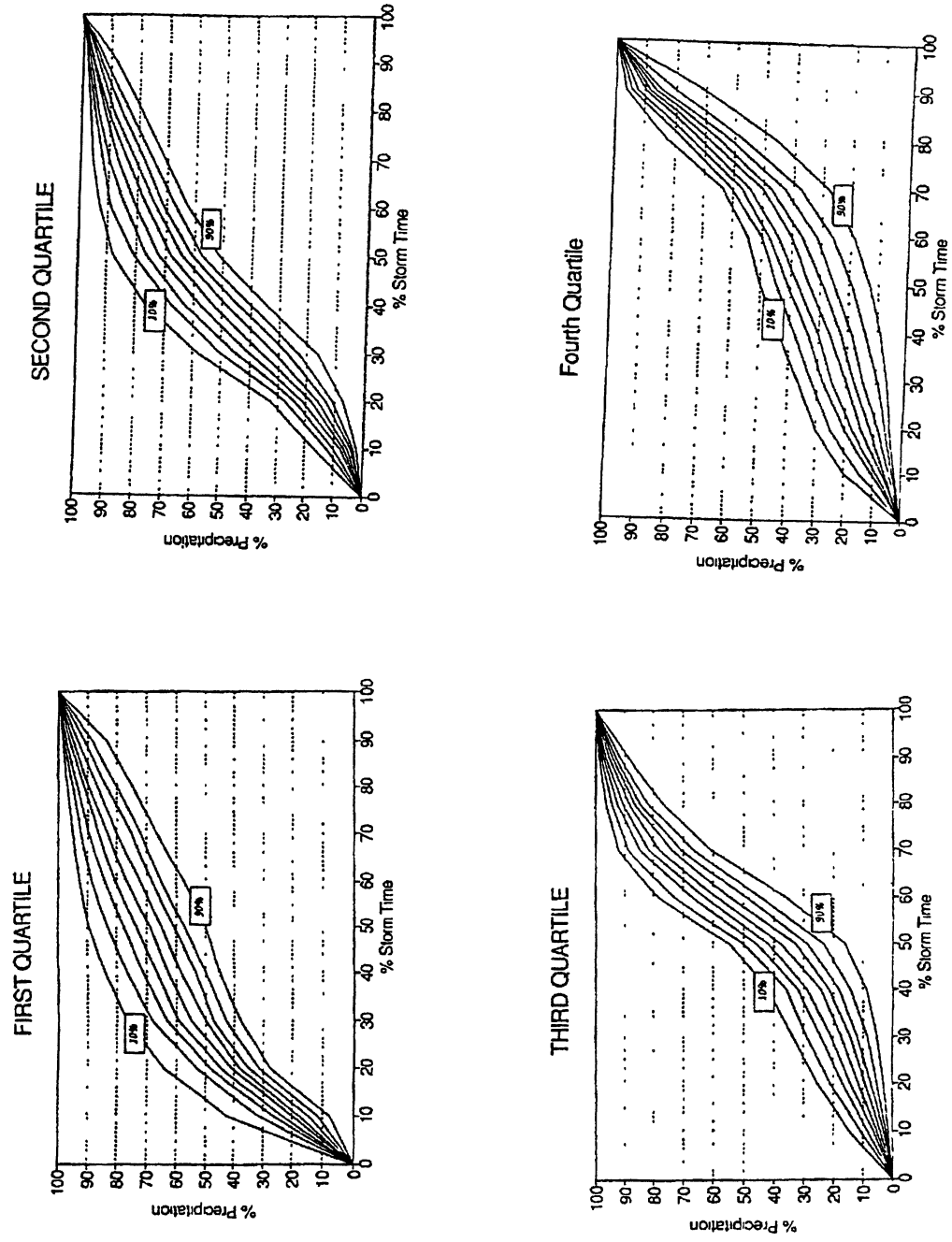
Table 2.1.8
80% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	11.59	3.33	2.12	2.93	12.40	2.92	2.22	2.78	11.01	3.43	2.82	2.94	11.43	3.83	3.24	3.57
20	31.43	10.43	5.26	6.25	33.33	9.80	5.17	6.36	32.00	10.61	6.06	5.89	30.67	10.73	7.45	7.69
30	42.82	22.56	8.45	9.09	45.83	22.02	8.70	10.22	42.86	22.22	9.16	9.12	41.00	22.05	10.97	11.11
40	50.00	40.00	14.29	13.85	51.06	40.00	13.60	13.50	50.00	39.19	14.29	12.92	49.23	40.22	16.67	16.00
50	57.14	57.14	23.33	18.52	59.14	56.73	24.00	17.89	56.34	56.00	25.00	16.67	55.67	55.56	26.74	21.06
60	64.84	66.67	44.00	26.00	65.37	66.67	43.48	27.29	64.29	66.41	45.09	24.85	63.22	65.00	46.11	26.32
70	71.76	75.00	66.67	37.04	72.00	75.56	66.33	38.33	71.43	74.32	67.11	35.00	70.20	73.08	66.67	36.47
80	80.00	83.33	81.94	56.64	80.00	84.72	80.70	55.56	80.00	82.92	80.95	54.29	77.14	80.43	80.00	54.78
90	88.57	92.00	91.60	80.00	89.71	92.50	91.67	77.73	88.18	91.43	90.77	78.71	86.67	90.00	89.33	79.00
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Table 2.1.9
90% Huff Curve Ordinates (Purdue et al., 1992)

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	7.78	2.07	1.37	2.13	7.69	1.80	1.27	1.61	7.50	2.16	1.89	1.82	8.00	2.48	2.12	2.08
20	27.50	7.07	3.13	4.40	28.89	6.48	3.39	3.87	27.27	7.14	3.77	3.89	27.27	7.54	4.65	4.55
30	37.50	16.07	5.21	6.12	38.79	16.67	5.65	6.25	38.46	15.11	6.00	5.45	35.17	16.32	7.50	7.25
40	45.00	34.58	8.70	8.78	46.15	34.29	9.52	8.92	45.57	33.00	10.14	7.69	42.86	34.69	12.43	10.53
50	50.00	51.09	16.00	12.67	51.22	51.90	13.71	11.65	50.00	50.00	19.23	10.00	50.00	50.00	20.51	13.89
60	57.27	62.22	35.71	18.52	60.00	62.00	34.00	20.00	58.57	62.00	36.11	15.56	59.11	60.00	40.00	20.00
70	66.67	70.85	60.83	27.50	66.67	71.11	61.75	30.60	66.67	69.81	61.67	25.60	66.67	67.50	62.22	28.57
80	74.67	79.07	76.96	45.67	74.29	80.00	76.67	46.32	75.00	78.67	77.00	45.00	72.14	76.67	76.10	45.71
90	83.53	88.00	88.35	70.00	84.00	90.00	88.64	69.27	84.00	88.21	88.13	70.83	82.00	86.00	85.71	70.91
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Figure 2.1.2
Huff Curves for Indianapolis (Purdue et al., 1992)



Example 2.1.1

This example problem illustrates the application of the Huff curves to generate a time distribution of rainfall.

If a storm in South Bend has a rainfall depth of 3.28 inches over a period of two hours, find the hyetograph (the time distribution of rainfall) and cumulative rainfall depths using a 50% Huff first quartile (Purdue et al., 1992) storm distribution.

Referring to Table 2.1.5, the 50% Huff first quartile curve for South Bend has the characteristics shown in the first and second columns below: Since the dimensionless time increment of the storm is 0.1 times the total storm times, the % storm times correspond to 0.1*120 minutes, or 12 minute durations as shown in column 3. The cumulative rainfall depth is found by multiplying the total rainfall depth by the cumulative percentage for that time. For example, at 70% storm time, the cumulative rainfall depth is $3.28 \times 75.98/100 = 2.49$ inches. These values are shown in column 4. The incremental rainfall values are obtained from the differences between the cumulative rainfall values. For example the incremental rainfall between 60 minutes and 72 minutes is $2.49 \text{ in.} - 1.92 \text{ in.} = 0.57 \text{ in.}$ and is shown in column 5. The cumulative and incremental results are shown graphically in Figures 2.1.3 and 2.1.4 respectively.

1	2	3	4	5
% Dimensionless Storm Time	South Bend 50% I- Quartile Dimensionless Storm Depth	Time (min)	Cumulative Rainfall (inches)	Incremental Rainfall (inches)
0	0.00	0	0.00	0.00
10	20.00	12	0.66	0.66
20	40.00	24	1.31	0.65
30	51.67	36	1.69	0.38
40	60.89	48	2.00	0.31
50	67.35	60	2.21	0.21
60	75.00	72	2.46	0.25
70	80.83	84	2.65	0.19
80	86.67	96	2.84	0.19
90	92.89	108	3.05	0.21
100	100.00	120	3.28	0.23

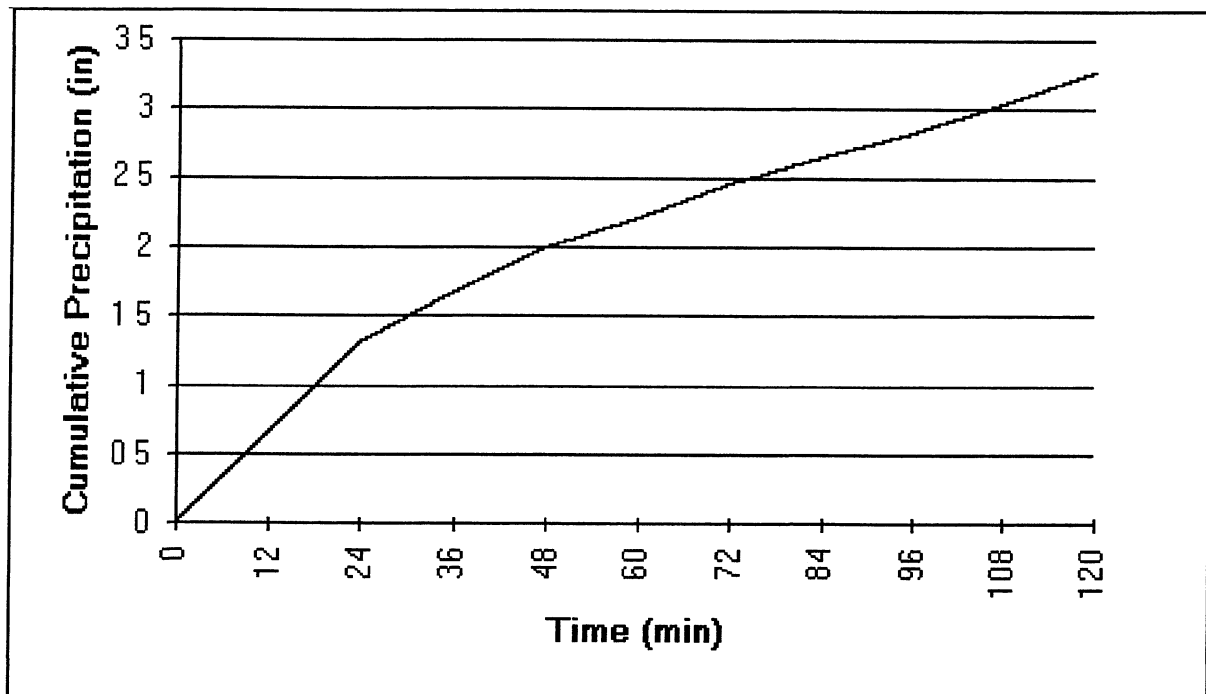


Figure 2.1.3
Cumulative Rainfall as a Function of Time, Example 2.1.1

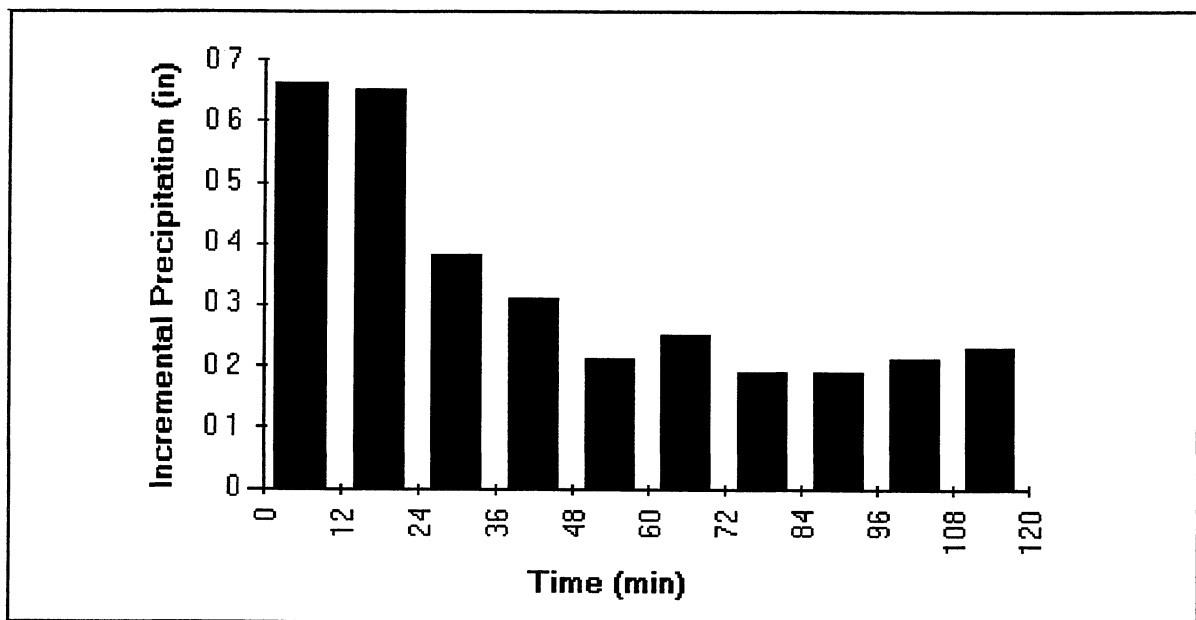


Figure 2.1.4
Incremental Rainfall as a Function of Time, Example 2.1.1

Table 2.1.10
Percentage Distribution of Quartiles for West Lafayette, Indiana
(Rao and Chenchayya, 1974)

Quartile	Ten Minute Data		Hourly Data	
	Number	Frequency	Number	Frequency
I	57	33	64	48
II	56	32	31	23
III	27	16	9	7
IV	34	19	29	22
Total	174	100	133	100

The time distribution of 174 rainfall events for the Lafayette region were analyzed by Rao and Chenchayya (1974). In Table 2.1.10 the frequency of occurrence for each quartile is shown. From the analysis it can be seen that the first and second quartile storms occur most frequently for both the ten-minute and hourly data.

The Soil Conservation Service (SCS) has also developed rainfall distributions. The SCS Type II 24-hour distribution is shown in Figure 2.1.5. This is the distribution commonly used by the SCS in planning and design. The application of this distribution is shown in Chapter 8. The SCS "B" rainfall distribution, which is a 6-hour distribution used in the development of emergency spillways, is shown in Figure 2.1.6.

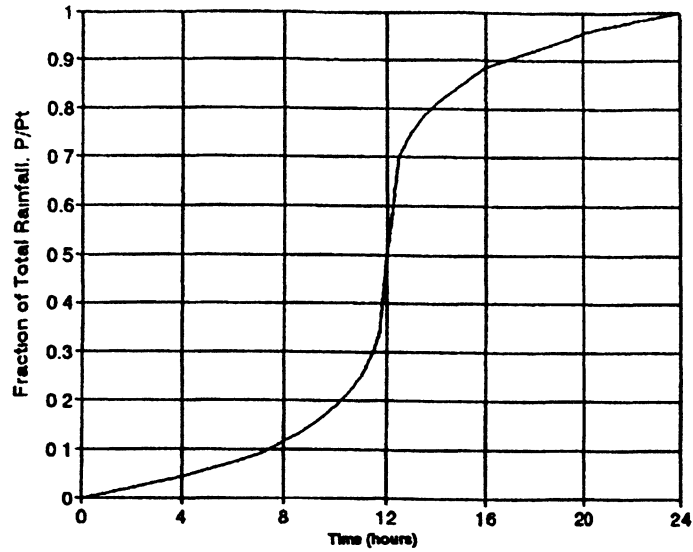


Figure 2.1.5 SCS "B" Rainfall Distribution (NEH-4)

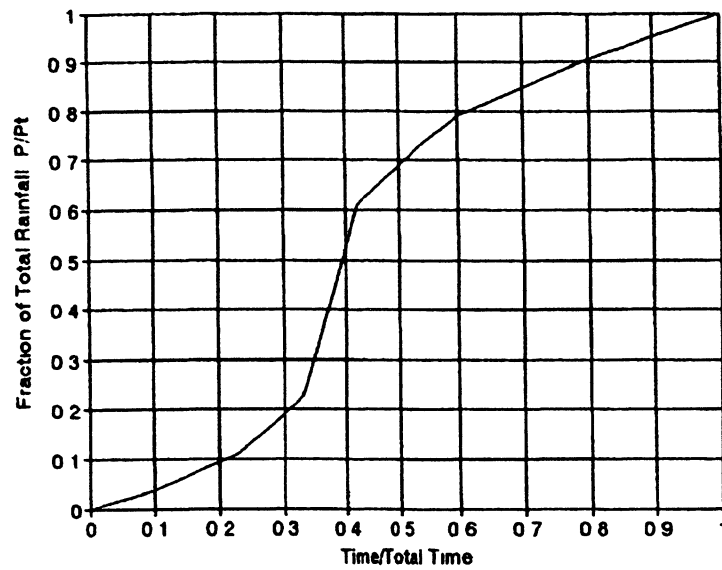


Figure 2.1.6 SCS Type II 24-hour Rainfall Distribution (NEH - 4)

2.1.4 - Areal Distribution of Rainfall

Rainfall measurements are made at specific locations in a watershed. The structure of a storm and its internal variation are not represented by a single point measurement or even by many point measurements. (Hershfield, 1961; Eagleson, 1970).

As the area represented by a point measurement increases, the reliability of the data as a representation of an average over the entire region decreases. As drainage areas become larger than a few square miles, point data must be adjusted to estimate areal rainfall. Figure 2.1.7 was developed by Hershfield (1961) and demonstrates the relationship between average rainfall and the point rainfall over a watershed as a function of area and storm duration. The use of Figure 2.1.7 is illustrated in Example 2.1.2.

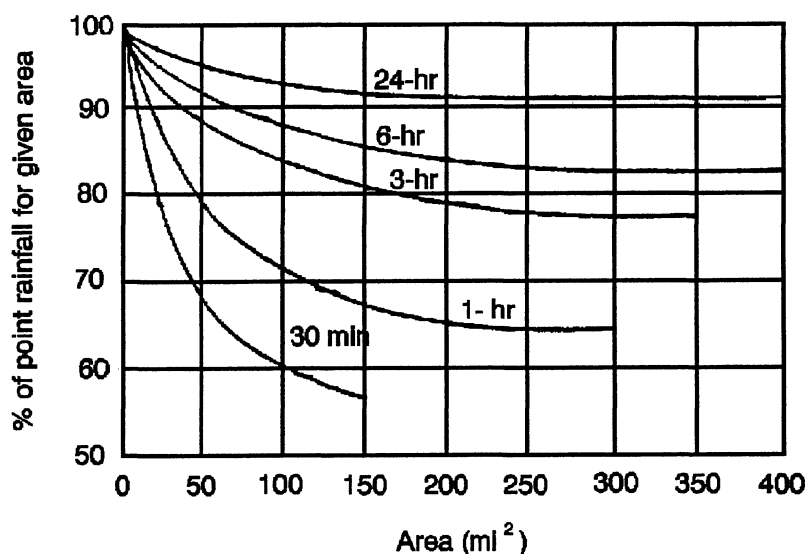


Figure 2.1.7 Area-Depth Curves (after Hershfield, 1961)

Example 2.1.2

A single gage is used to measure the rainfall over a 25 square mile watershed. If the gage collected 2.00 inches over a period of 3-hours, what is the estimated average rainfall over the entire watershed?

Referring to Figure 2.1.7, the 3-hour curve intersects a watershed area of 25 square miles at the ordinate 93%. Therefore the average rainfall over the entire watershed is $2.00 \times 0.93 = 1.86$ in.

2.2 - COLLECTION AND ANALYSIS OF PRECIPITATION DATA

Drainage design engineers usually do not collect and analyze precipitation data. Rather, they use data that is already published. Information developed in this section will aid the designer to interpret the data published in sources such as the United States Weather Bureau (now known as the National Weather Service) Technical Paper Number 40 and NOAA Technical Memo NWS HYDRO-35. This information is also beneficial in understanding concepts presented in other chapters of this manual.

2.2.1 - Precipitation Measurement and Interpretation

Variation in rainfall depths over an area is determined from rainfall depths observed at selected points in the watershed. Unless the rain gage density is high, accurate estimation of rainfall pattern and average values of rainfall depths usually cannot be obtained.

Rainfall is recorded with different levels of accuracy. First, there are the first-order Weather Service stations. These gages produce a continuous time-depth sequence which is usually transferred to an hourly sequence. Second, there are the recording-gage data of the hydrologic network which are published for clock-hour intervals. These data are processed to get hourly data. Thirdly, there are a very large number of nonrecording-gages, to obtain daily rainfall depths. (Hershfield, 1961).

After the data from particular gaging stations over an area have been collected, it may be necessary to average the depth of precipitation over an area. There are three methods of computing this average.

The first and the simplest method of obtaining the average depth is by using the arithmetic average. This method yields good estimates if the terrain is flat, and the gages are uniformly distributed and the individual gage catches do not vary widely from the mean.

In the second method, known as the Thiessen method, each gage is given a weight. The station locations are drawn on a map, and lines connecting the stations are drawn. Perpendicular bisectors of these connecting lines form polygons around each station. The area of each polygon is determined by planimetry and is expressed as a percentage of the total area. Weighted average rainfall for the total area is computed by multiplying the precipitation at each station by its assigned percentage of area and summing them up. The results from this method are regarded as more reliable than those obtained by simple arithmetic averaging.

The third and the most accurate method of estimating the average precipitation over a watershed is the isohyetal method. Station locations and amounts are plotted on a suitable map, and contours of equal precipitation (isohyets) are drawn. The average precipitation over an area is computed by multiplying the average precipitation between successive isohyets by the area of the watershed located between these isohyets, totaling these products, and then

the area of the watershed located between these isohyets, totaling these products, and then dividing the sum by the total area. The isohyetal method permits the use and interpretation of all available data and reflects orographic effects and storm distribution. Sample calculations of the three methods are shown in Figure 2.2.1

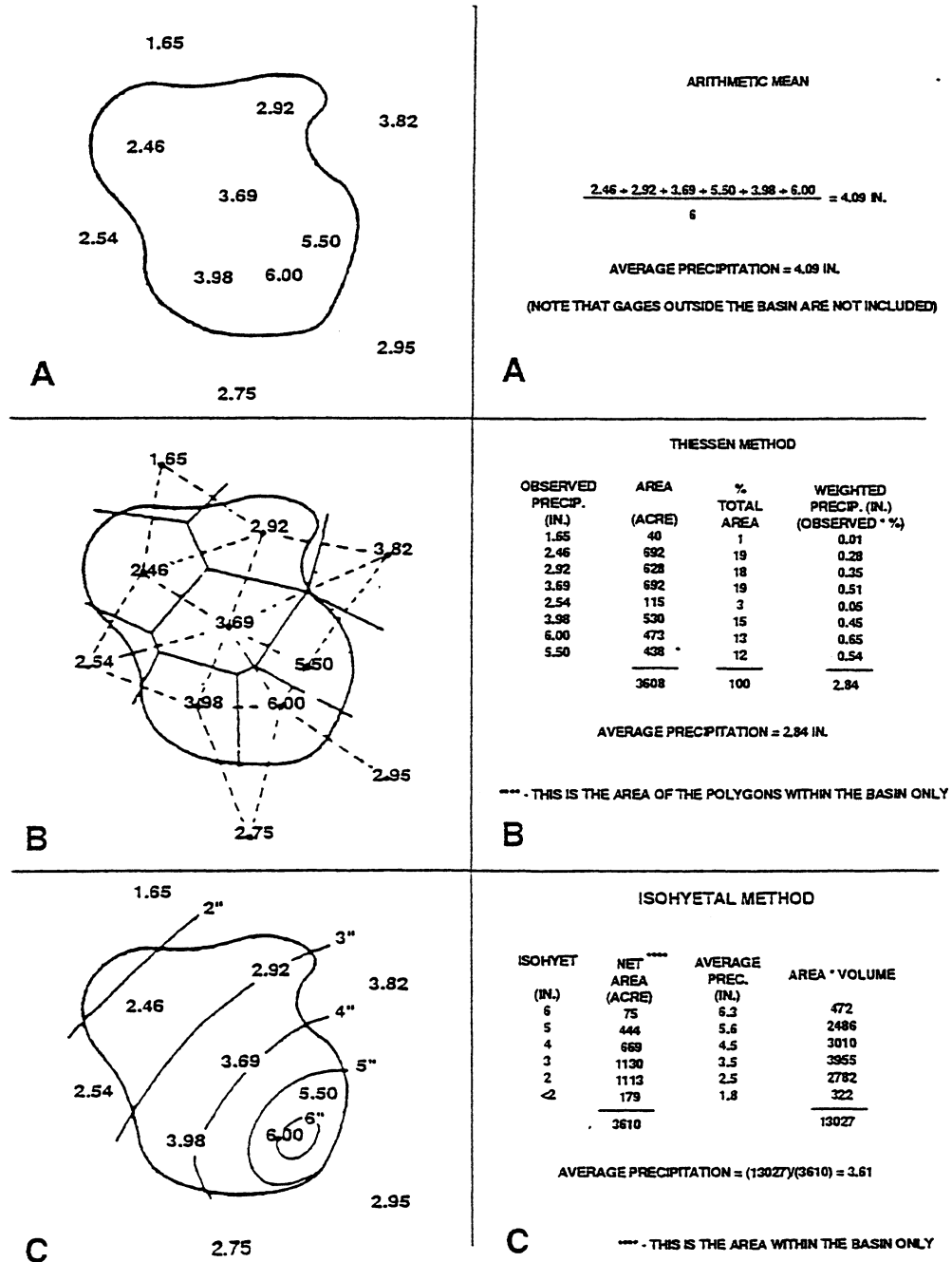


Figure 2.2.1 Areal Averaging of Precipitation by (a) Arithmetic Method (b) Thiessen Method (c) Isohyetal Method (Linsey et al., 1975)

2.2.2 - Statistical Analysis of Rainfall Data

Analysis of rainfall records to obtain design data is sometimes necessary. When more detailed information for a specific location is required, extreme value analysis is required. For this type of study, no less than 20 years of record is required if the approach is to have any statistical reliability. A brief discussion of statistical analysis of extreme rainfall data is presented in Appendix A.

The rainfall depth occurring over a prespecified duration is a basic unit of information used in drainage design. This information is used to estimate runoff from watersheds by using the techniques presented in Chapter 3. The selection of a design frequency is based on economic analysis and policy decisions.

Since theoretical aspects of frequency analysis for extreme values require that all data for the period of study be comparable, it is important that the basic data be thoroughly scrutinized. Data are comparable when all of it represents accurate, reliable observations. Data from individual storms are assumed to be independent. The following analysis must be applied only to extreme values. Changes in the location of a gage or other extraneous effects should be corrected.

The annual maximum series consists of only the largest value in any given year. The data are arranged in descending order of magnitude and assigned a rank (m) starting with one and increasing by one until the rank number equals the number of observations (n). For locations with limited data, the partial duration series may be more appropriate. The partial duration series consist of the n values larger than a threshold value regardless of the year of the storm event.

The return periods for the particular set of data are calculated by using Equation 2.2.1.

$$T_r = \frac{n + 1}{m} \quad (2.2.1)$$

where T_r is the return period of n -year event, n is the number of events or years of record, and m is the order or ranking number.

The return period may be transposed to frequency by using Eq. 2.2.2:

$$P = \frac{1}{T_r} = \frac{m}{1 + n} \quad (2.2.2)$$

where P = the frequency (average probability of occurrence in a year) of the event being equalled or exceeded.

For example, if it was found that three inches of rain in a given duration fell once in a nineteen year period of record, one might state that on the average, three inches of rainfall would occur once every twenty years or with a frequency of $P = 1 / (19+1) = 0.05$.

Due to the small number of observations which are usually available, methods are needed to estimate rainfall magnitudes corresponding to larger return periods. Many well-defined theoretical probability distributions have been used to estimate rainfall magnitude at large return periods. It should be emphasized, however, that any theoretical distribution is not an exact representation of the natural process, but is only a probability description of the probabilistic structure of the process. Two of the commonly used distributions for rainfall analysis are the Gumbel's extreme value distribution and the Log Pearson Type III distribution. Either may be used as a formula or as a graphical approach. The general equation for each is Equation 2.2.3 (Chow et al., 1988),

$$p = \bar{p} + Ks \quad (2.2.3)$$

where p is the desired peak value for a specific frequency, \bar{p} = arithmetic average of the given rainfall data, K is the frequency factor (use K for Gumbel distribution from Table 2.2.1 or K_L for Log Pearson Type III distribution from Table 3.5.2), and s is the standard deviation of the given rainfall data

In utilizing Gumbel's distribution, the arithmetic average in Eq. 2.2.4 is used:

$$\bar{p} = \frac{1}{n} \sum_{i=1}^n p_i \quad (2.2.4)$$

where p_i is the individual extreme value of rainfall and n is the number of events or years of record.

The standard deviation is calculated by Eq. 2.2.5

$$s = \left[\frac{1}{n-1} \sum_{i=1}^n (p_i - \bar{p})^2 \right]^{\frac{1}{2}} \quad (2.2.5)$$

The frequency factor (K) (given in Table 2.2.1), which is a function of the return period and sample size, when multiplied by the standard deviation gives the departure of a desired return period rainfall from the average

The Log Pearson Type III (LP (III)) distribution involves logarithms of the measured values. The mean and the standard deviation are determined using the logarithmically transformed data. The simplified expression for this distribution is given as (see also Equation A-12 in

Appendix A) (Chow et al , 1988)

$$\log p = \bar{y} + K_t s_y \quad (2.2.6)$$

$$\text{where} \quad \bar{y} = \frac{1}{n} \sum_{i=1}^n y_i \quad (2.2.7)$$

$$\text{where} \quad y_i = \log(p_i) \quad (2.2.8)$$

$$s_y = \left(\frac{1}{n-1} \sum_{i=1}^n (y_i - \bar{y})^2 \right)^{\frac{1}{2}} \quad (2.2.9)$$

The skewness coefficient, C_s , is required to compute the frequency factor for this distribution. The skewness coefficient is computed by Eq 2.2.10 (Chow et al , 1988)

$$C_s = \frac{\frac{n}{n-1} \sum_{i=1}^n (y_i - \bar{y})^3}{(n-2) s_y^3} \quad (2.2.10)$$

By knowing the skewness coefficient and the recurrence interval, the frequency factor, K_t for the LP(III) distribution, is read off from Table 3.5.2. The antilog of the solution in Equation 2.2.6 will provide the estimated extreme value for the given return period.

Table 2.2.1
Frequency Factors (K) for the Gumbel Distribution

Sample Size	Recurrence Interval						
	10	20	25	50	75	100	1000
15	1.703	2.410	2.632	3.321	3.721	4.005	6.265
20	1.625	2.302	2.517	3.179	3.563	3.836	6.006
25	1.575	2.235	2.444	3.088	3.463	3.729	5.842
30	1.541	2.188	2.393	3.026	3.393	3.653	5.727
40	1.495	2.126	2.326	2.943	3.301	3.554	5.476
50	1.466	2.086	2.283	2.889	3.241	3.491	5.478
60	1.446	2.059	2.253	2.852	3.200	3.446	
70	1.430	2.038	2.230	2.824	3.169	3.413	5.359
75	1.423	2.029	2.220	2.812	3.155	3.400	
100	1.401	1.998	2.187	2.770	3.109	3.349	5.261

Example 2.2.1

Using the twenty-five years of data tabulated below, determine the 10-year and 50-year precipitation depths for a 1-hour duration storm in Coshocton, OH. Assume that the Gumbel distribution is applicable (adapted from Chow et al., 1988).

<u>Rank</u>	<u>Max. Depth (in.) for 60-min duration</u>	<u>Rank</u>	<u>Max. Depth (in.) for 60-min duration</u>
1	3 220	13	1 204
2	1 830	14	1 203
3	1 756	15	1 200
4	1 510	16	1 194
5	1 431	17	1 192
6	1 375	18	1 174
7	1 313	19	1 143
8	1 306	20	1 130
9	1 290	21	1 130
10	1 269	22	1 109
11	1 225	23	1 095
12	1 213	24	1 094
		25	1 063

Using Equations 2 2 4 and 2 2 5, the sample statistics are

$$\bar{P} = \frac{1}{n} \sum_{i=1}^n P_i = \frac{1}{25} (3.220 + 1.830 + \dots + 1.063) = 1.347$$

$$S = \left(\frac{1}{n-1} \sum_{i=1}^n (P_i - \bar{P})^2 \right)^{\frac{1}{2}}$$

$$= \left(\frac{1}{24} ((3.220 - 1.347)^2 + (1.830 - 1.347)^2 + \dots + (1.063 - 1.347)^2) \right)^{\frac{1}{2}}$$

$$S = 0.434$$

The rainfall depth is obtained by

$$P_t^{T_r} = \bar{P} + K S$$

where T_r is the return period (years) and t is the time (hours).

From Table 2 2 1, for a 10-year return period with sample size equal to 25, K is 1.575 From Table 2 2 1, for a 50-year return period with sample size equal to 25, K is 3.088

$$p_1^{10} = 1.347 + 1.575(0.434) = 2.031 \text{ inches}$$

$$p_1^{50} = 1.347 + 3.088(0.434) = 2.687 \text{ inches}$$

2 2 3 - Hydraulic Risk

A return period of one hundred years implies that on the average that event will occur or be exceeded once every one hundred years, but does not guarantee that the event will occur every one hundred years. The concept of risk takes this into account by considering the chance of a particular event occurring within a given period. This risk is often something that the engineer must use in determining the economic feasibility of a design. Risk is determined by Eq 2 2 11

$$J = 1 - (1 - P)^n = 1 - \left(1 - \frac{1}{T_r}\right)^n \quad (2 2 11)$$

where J is the risk of a certain event during a time interval, T_r is the return period, n is the number of years in the time interval

Example 2.2.2

What is the risk of exceeding a 10-year return period storm in the next 5 years?

$T_r = 10$ and $n = 5$

$$J = 1 - \left(1 - \frac{1}{T_r}\right)^n = 1 - \left(1 - \frac{1}{10}\right)^5 = 0.41$$

So, the risk of exceeding this storm in the next 5 years is 41%.

2.2.4 - Depth and Intensity-Duration-Frequency Equations

Curves of depth or intensity-duration-frequency have been developed for various stations of Indiana (Purdue et al., 1992). These are very useful to the design engineer as input to the deterministic runoff models. To ease computational effort and in order to incorporate these curves in computer models, equations have been developed. An understanding of these equations or curves will aid the designer in their use for specific locations.

Depth and intensity-duration-frequency curve distributions are developed by using distributions as discussed in the last section. Depth and intensity are related, since intensity, i , is nothing more than depth, ΔD , divided by an increment of time, Δt , as shown in Equation 2.2.12.

$$i = \frac{\Delta D}{\Delta t} \quad (2.2.12)$$

It is important to realize that the intensity-duration-frequency values obtained do not represent any particular storm pattern or storm. It is the maximum amount of rain that has fallen for a particular time interval over the n -years of record.

The rainfall intensities, i , corresponding to a storm duration, t (hours), and a recurrence interval, T_r , can be represented in the form:

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} \quad (2.2.13)$$

where c , d , α , and β are regional coefficients determined by evaluation of rainfall intensity-duration-frequency curves. The coefficients and exponents for several major cities in Indiana have been calculated by Purdue et al., 1992 and are shown in Table 2.2.2. The curves generated by this equation, for Indianapolis, are shown in Figure 2.2.2. In order to produce a smooth curve, curve fitting was used between the 0.6 hour and 2.0 hour values. Equation 2.2.13 is referred to as the intensity-duration-frequency (IDF) equation.

Example 2.2.3

Using the IDF equation, determine the 10-year, 15-minute rainfall intensity for the City of Indianapolis.

From Table 2.2.2, $c=2.1048$ $\alpha=0.1733$ $d=0.470$ $\beta=1.1289$

Referring to the IDF equation,

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{2.1048 (10)^{0.1733}}{(\frac{15}{60} + 0.470)^{1.1289}} = 4.545 \text{ inches/hour}$$

Table 2.2.2
Regional Coefficients for the IDF Equation (Eq. 2.2.13)
(Purdue et al., 1992)

Station	c	α	d	β
0.083 hour < t ≤ 1 hour				
Indianapolis	2.1048	0.1733	0.470	1.1289
South Bend	1.7204	0.1753	0.485	1.6806
Evansville	1.9533	0.1747	0.522	1.6408
Fort Wayne	2.0030	0.1655	0.516	1.4643
1 hour < t < 36 hour				
Indianapolis	1.5899	0.2271	0.725	0.8797
South Bend	1.2799	0.1872	0.258	0.8252
Evansville	1.3411	0.2166	0.300	0.8154
Fort Wayne	1.4381	0.1878	0.525	0.8616

IDF Curves for Indianapolis

Return Period - 2,5,10,20,50 & 100yrs

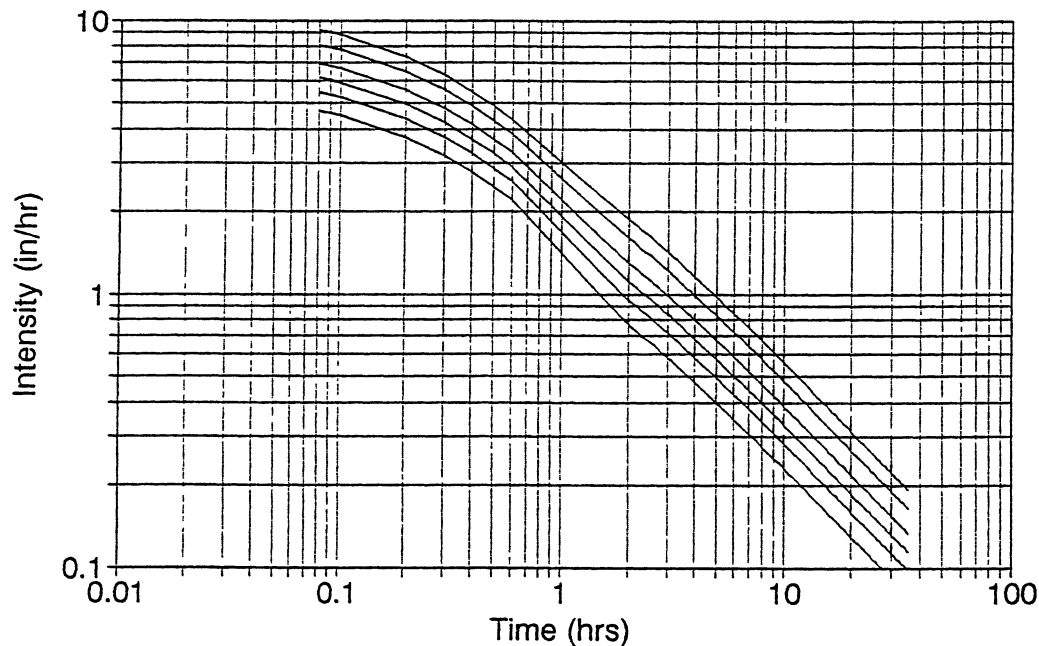


Figure 2.2.2

Intensity - Duration - Frequency Relationship for Indianapolis by IDF Equation
(adapted from Purdue et al., 1992)

2.2.5 Chen's Method

The coefficients and exponents in Table 2.2.3 are not available for all locations of interest to a drainage engineer. In such cases the rainfall intensity-duration-frequency data for a site will have to be estimated. An accurate method to generate this intensity-duration-frequency information has been developed by Chen (FHWA, 1976). This method has been shown to be valid across the United States. Maps showing rainfall depth curves across the entire continental United States are located at the end of Appendix A. Figures 2.2.4 through 2.2.6 enable rainfall depths in Indiana to be determined for Chen's method. The procedure is as follows:

1. From the rainfall atlases in Figures 2.2.4-2.2.6, estimate the following:
 - a. The 10-year, 1-hour rainfall depth, p_1^{10}
 - b. The 10-year, 24-hour rainfall depth, p_{24}^{10}
 - c. The 100-year, 1-hour rainfall depth, p_1^{100}
2. Compute the following parameters:
 - a. $(p_1^{10}) / (p_{24}^{10})$
 - b. $X = (p_1^{100}) / (p_1^{10})$

3. Estimate a_1 , d' , and n' from Figure 2.2.3.
4. Determine the intensity corresponding to the 10-year, 1-hr rainfall depth, i_1^{10} , which is equal to p_1^{10} .
5. Calculate the parameter a' , where a' is defined below:

$$a' = a_1 i_1^{10} \log(10^{(2-x)} T_r^{(x-1)})$$

where T_r is the recurrence interval in years.

6. Determine the rainfall intensity-duration-frequency formula:

$$i_t^{T_r} = \frac{a'}{(60t + d')^{n'}} \quad (\text{inches/hour}) \quad (2.2.14)$$

$$i_t^{T_r} = \frac{2.54 a'}{(60t + d')^{n'}} \quad (\text{cm/hr}) \quad (2.2.15)$$

where the time, t , has units of hours and the intensity is $i_t^{T_r}$.

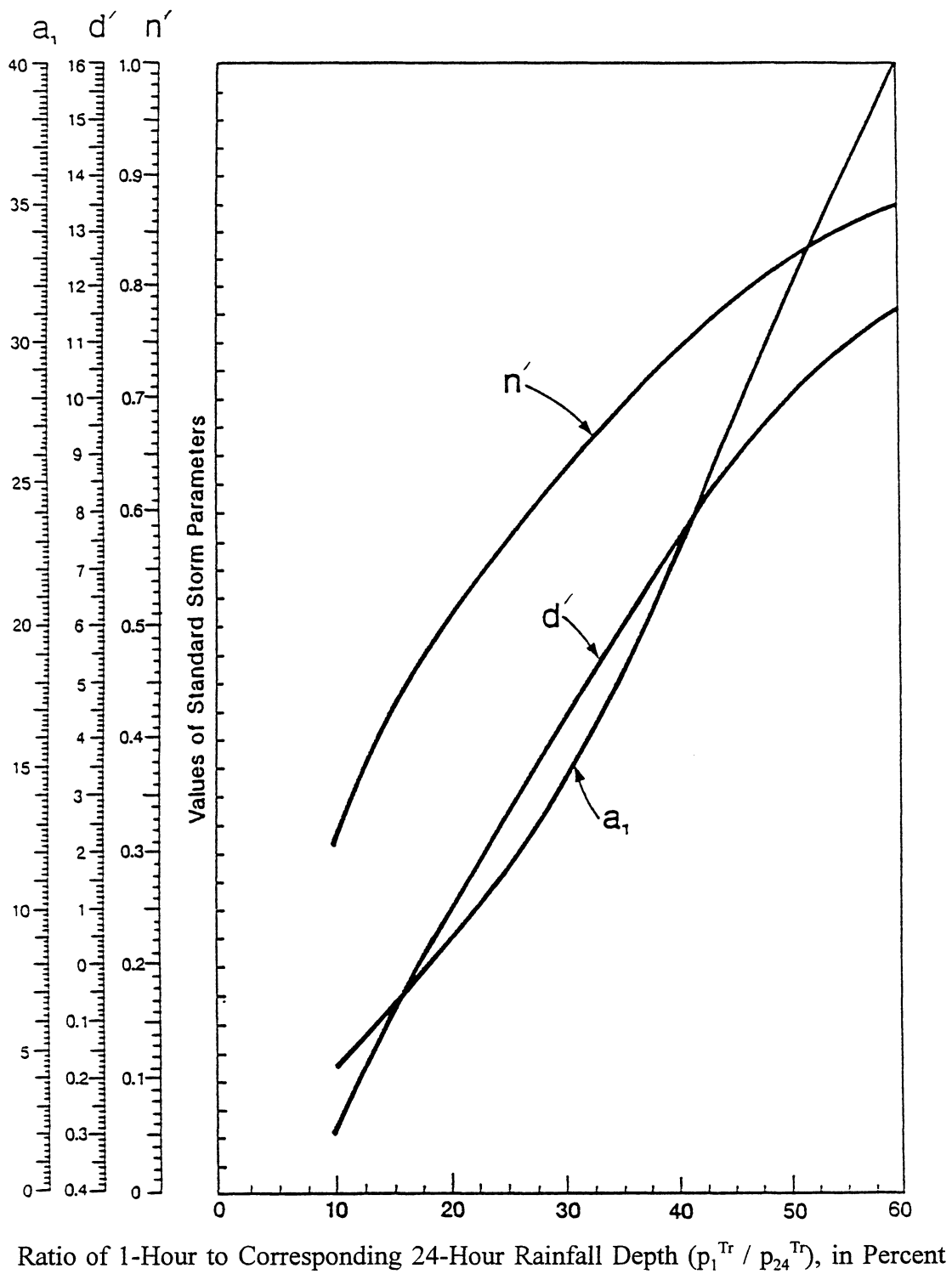


Figure 2.2.3
Chen's Method Parameters as a Function of p_1^{10}/p_{24}^{10}

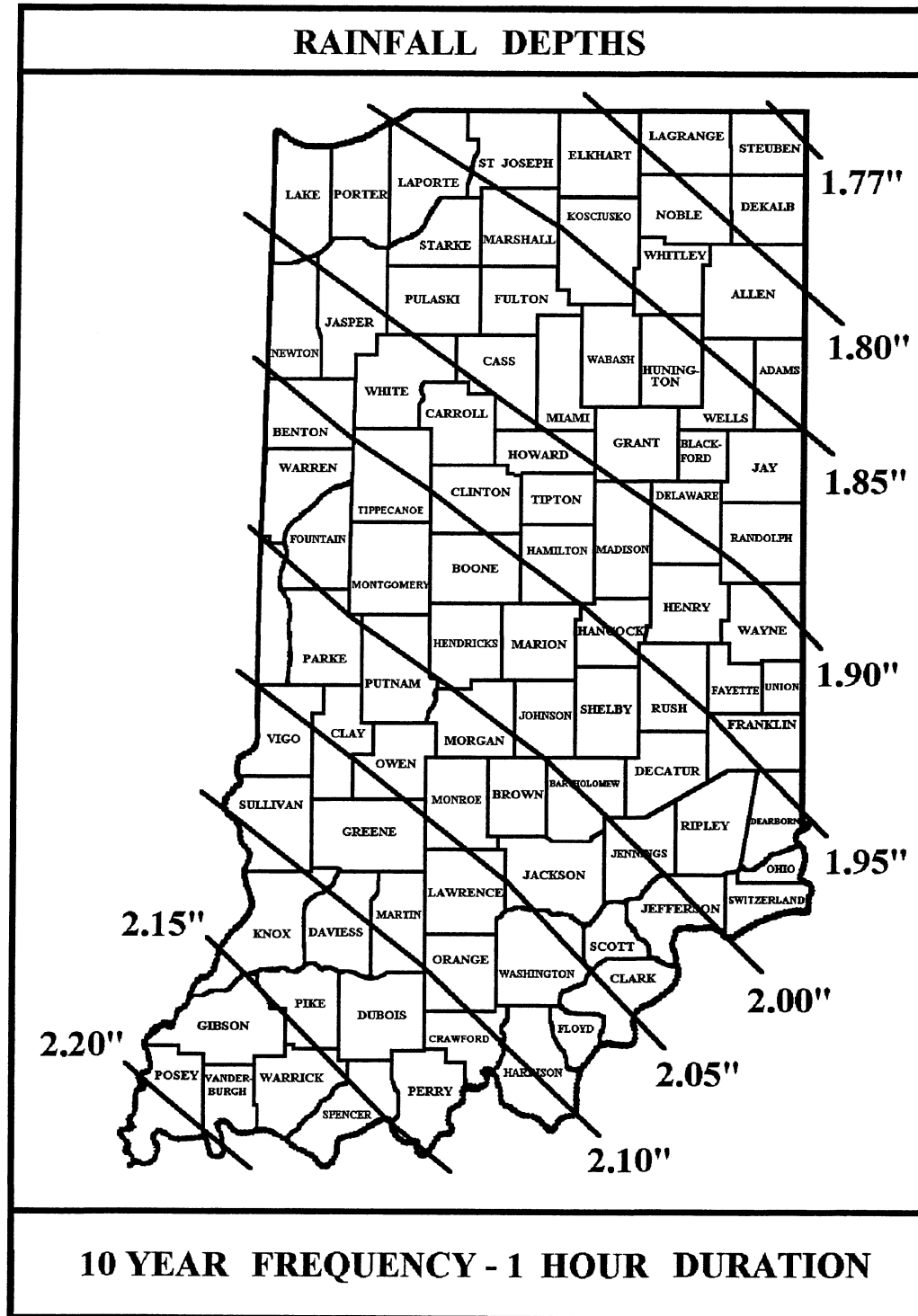


Figure 2.2.4
Depth - Duration - Frequency Curves
(adapted from Technical Paper 40, 1961)

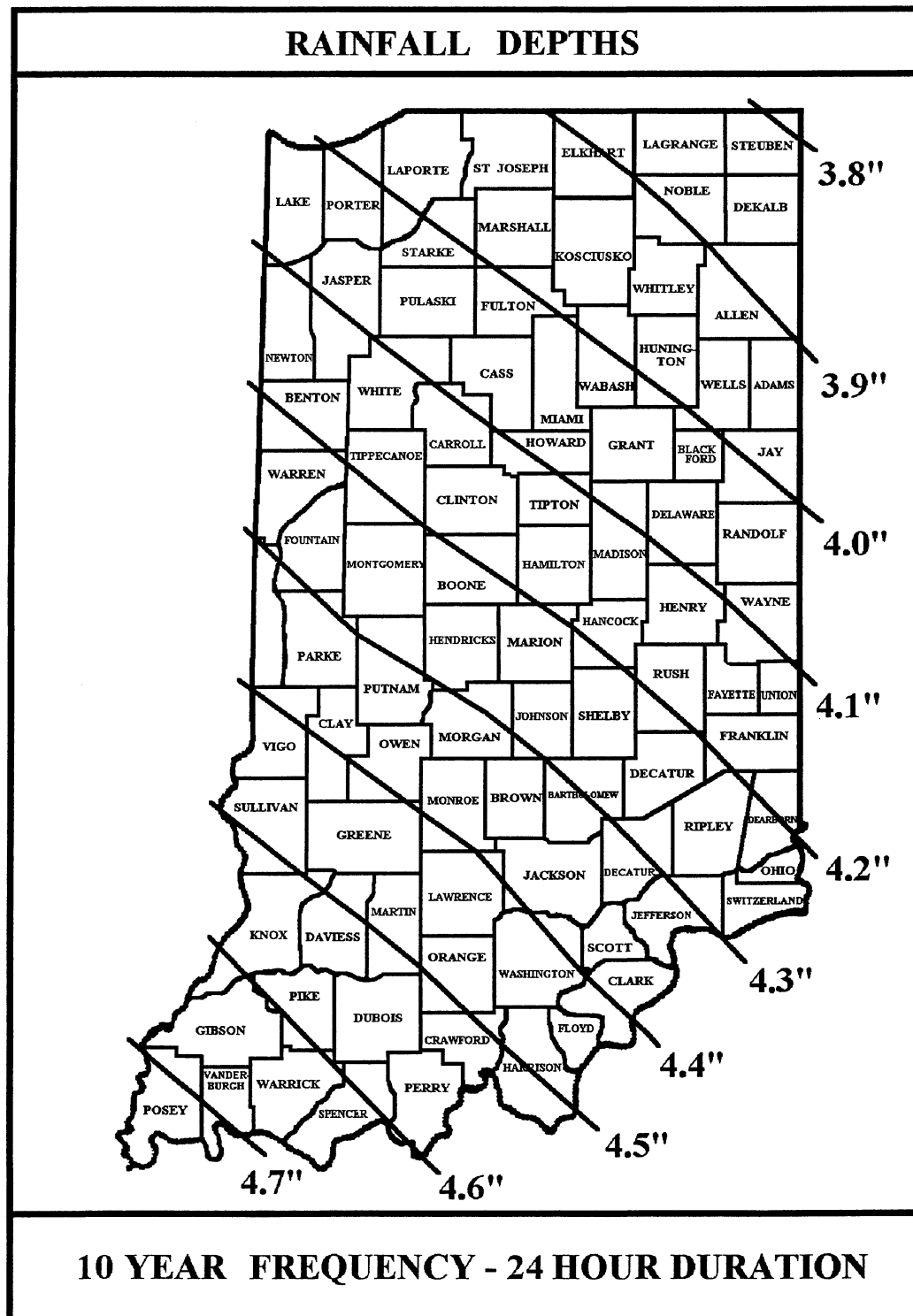


Figure 2.2.5
Depth - Duration - Frequency Curves
(adapted from Technical Paper 40, 1961)

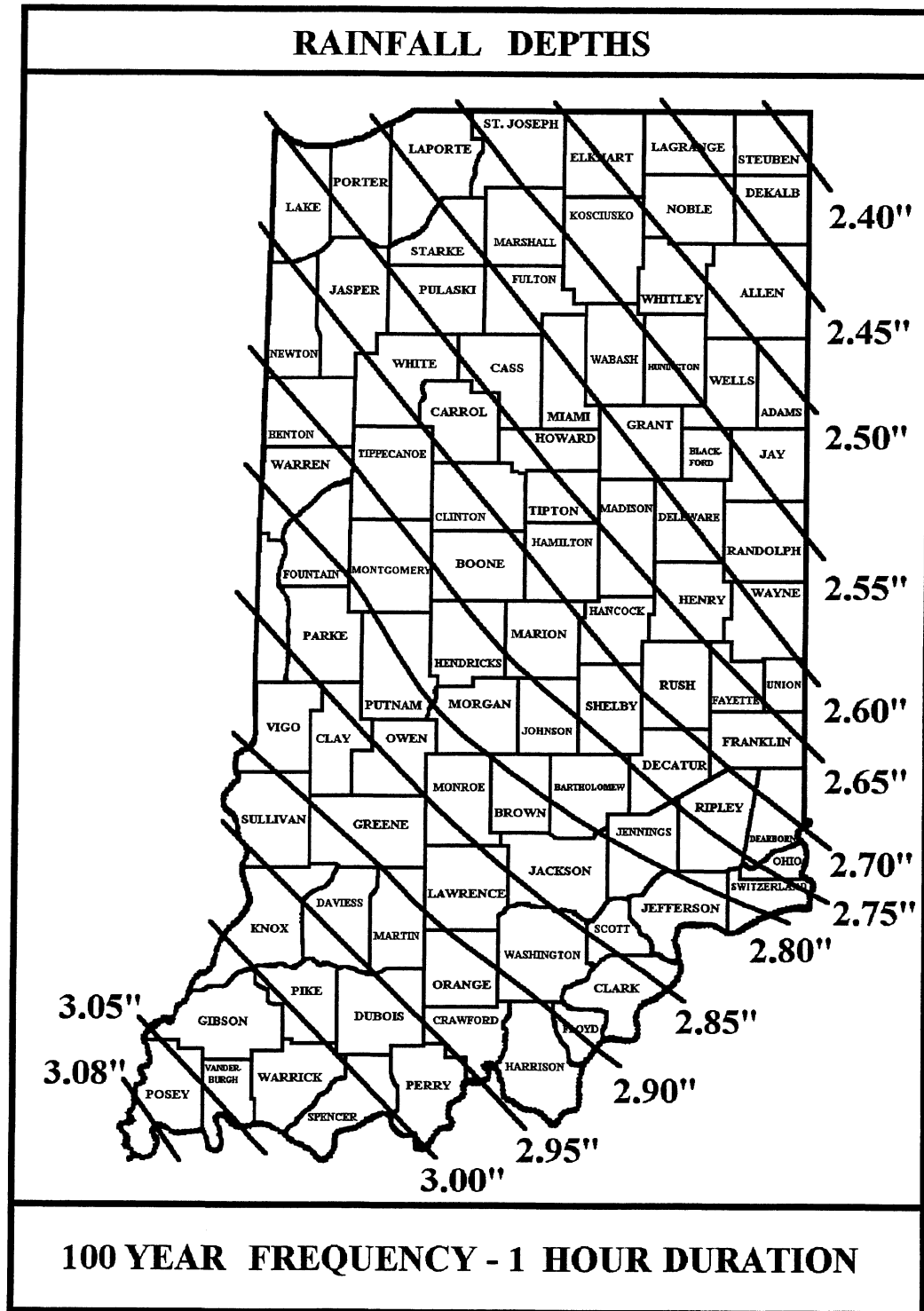


Figure 2.2.6
Depth - Duration - Frequency Curves
(adapted from Technical Paper 40, 1961)

Example 2.2.4

For the city of Indianapolis, compute the 10-year and the 100-year intensity-duration equations (i_t^{Tr} in units of in/hr) using Chen's method.

STEP 1

Obtain: The 10-year, 1-hour rainfall depth, p_1^{10}
 The 10-year, 24-hour rainfall depth, p_{24}^{10}
 The 100-year, 1-hour rainfall depth, p_1^{100}

Using an engineer's scale and Figures 2.2.4 - 2.2.6,

$$p_1^{10} = 1.97''$$

$$p_{24}^{10} = 4.23''$$

$$p_1^{100} = 2.73''$$

STEP 2

Compute:

$$(p_1^{10}) / (p_{24}^{10}) = 1.97'' / 4.23'' = 0.466$$

$$X = (p_1^{100}) / (p_1^{10}) = 2.73'' / 1.97'' = 1.386$$

STEP 3

Read off a_1 , d' , and n' from Figure 2.2.3.

$$a_1 = 28.6$$

$$d' = 9.2$$

$$n' = 0.78$$

STEP 4

Determine i_1^{10} .

$$i_1^{10} = p_1^{10} / 1 \text{ hr} = 1.97'' / \text{hr}$$

STEP 5

Compute a' :

$$a' = a_1 i_1^{10} \log(10^{(2-X)} T_r^{(X-1)})$$

$$a' = (28.6)(1.97) \log(10^{(2-1.386)} T_r^{(1.386-1)})$$

$$a' = 56.34 \log(4.11 T_r^{(0.386)})$$

STEP 6

Determine the rainfall intensity-duration-frequency formulae for Indianapolis:

$$i_t^{T_r} = \frac{a'}{(60t + d')^{n'}} \quad (\text{inches/hour})$$

$$i_t^{T_r} = \frac{56.34 \log(4.11 T_r^{0.386})}{(60t + 9.2)^{0.78}} \quad (2.2.16)$$

$$i_t^{10} = \frac{56.34 \log(4.11 10^{0.386})}{(60t + 9.2)^{0.78}} \quad \text{10-year intensity-duration equation}$$

$$\therefore i_t^{10} = \frac{56.33}{(60t + 9.2)^{0.78}}$$

$$i_t^{100} = \frac{56.34 \log(4.11 100^{0.386})}{(60t + 9.2)^{0.78}} \quad \text{100-year intensity-duration equation}$$

$$\therefore i_t^{100} = \frac{78.08}{(60t + 9.2)^{0.78}}$$

Substitute time values (in hours) to get intensities in inches/hour. For intensities in cm/hr, simply multiply the right hand side of the equation by 2.54 cm/in.

Example 2.2.5

Compare the rainfall intensities obtained for Indianapolis using the IDF equation to those using Chen's method. Use a 100-year recurrence interval.

From Table 2.2.2:

$$c=2.1048 \quad \alpha=0.1733 \quad d=0.470 \quad \beta=1.1289 \\ \text{for } 0.083 \text{ hours} < t \leq 1 \text{ hour}$$

$$c=1.5899 \quad \alpha=0.2271 \quad d=0.725 \quad \beta=0.8797 \\ \text{for } 1 \text{ hour} < t < 36 \text{ hours}$$

Therefore, using the IDF equation with a recurrence interval of 100 years:

$$i_t^{100} = \frac{2.1048 (100)^{0.1733}}{(t + 0.470)^{1.1289}} \quad 0.083 \text{ hours} < t \leq 1 \text{ hour}$$

$$i_t^{100} = \frac{1.5899 (100)^{0.2271}}{(t + 0.725)^{0.8797}} \quad 1 \text{ hour} < t \leq 36 \text{ hr}$$

Where t has units of hours in the above equations.

From Example 2.2.4, the 100-year rainfall intensity for Indianapolis, i_t^{100} , is:

$$i_t^{100} = \frac{78.08}{(60t + 9.2)^{0.78}}$$

Where t has units of hours in the above equation.

Intensities according to both methods are summarized below for Indianapolis:

Time (hours)	Intensity (in/hr) by IDF Equation 2 2 13	Intensity (in/hr) by Chen's Method
0.1	8.82	9.35
0.5	4.84	4.46
1	3.03	2.87
2	1.87	1.76
3	1.42	1.31
4	1.15	1.05
5	0.97	0.89
6	0.85	0.78
8	0.67	0.62
10	0.56	0.53
12	0.48	0.46
15	0.40	0.38
20	0.31	0.31
24	0.27	0.27

Chen's method gives results that compare well with those of Equation 2.2.13, as shown in Figure 2.2.7.

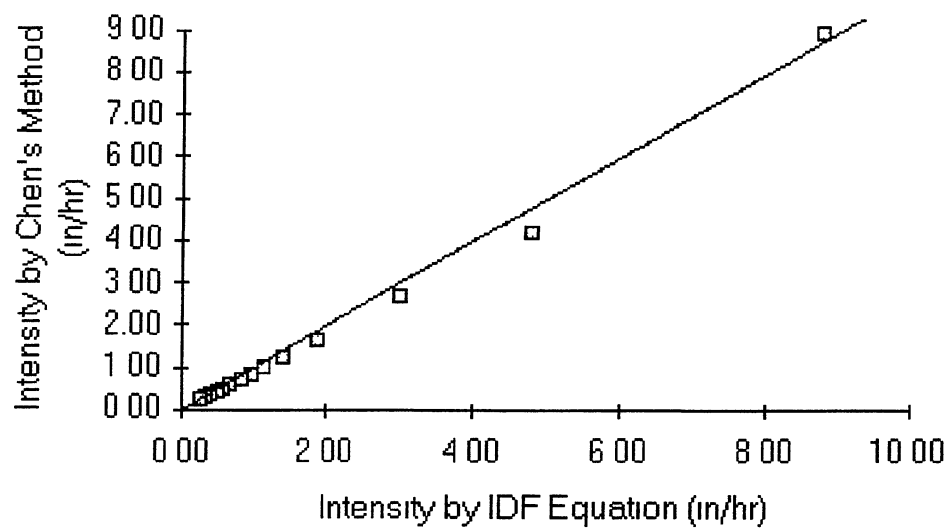


Figure 2.2.7
Relationship Between IDF Equation and Chen's Method for
Indianapolis, 100-Year Recurrence Interval from Example 2.2.5

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Chapter 3

RUNOFF AND ITS ESTIMATION

Chapter 3 - RUNOFF AND ITS ESTIMATION

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LIST OF PARAMETERS

A	Area (acres)
A'	Area (ft ² or m ²)
A _m	Area (mi ²)
B	Coefficient for Izzard's t _c formula
c,d,α,β	Regional coefficients for determining rainfall intensity-duration-frequency curves
C	Rational method runoff coefficient
c'	Retardance coefficient for Izzard's t _c formula
CN	Curve number
C _m	Map skew
C _s	Coefficient of skewness
C _w	Weighted skew
C _{comp}	Composite runoff coefficient
CN _{comp}	Composite curve number
DA	Drainage area (mi ²)
ΔD	Rainfall duration
F(t)	Total infiltration
G	WRC method parameter
H	WRC method parameter
i	Rainfall intensity (in/hr or cm/hr)
I _a	Initial abstraction
K	Coefficient for t _c equations
K _n	Coefficient used in outlier detection equation
K _T	Coefficient used in WRC method
L	Length (ft or m)
L'	Watershed lag
L _m	Length (miles)
μ	max [1, t+1 - NU]
n	Number of data values
n	Manning's roughness coefficient
N	Kerby's retardance coefficient
NP	Number of elements in the rainfall hyetograph up to last non-zero entry
NU	Number of coordinates in the unit hydrograph up to last non-zero entry
v	min [t, NP]
P(t)	Accumulated rainfall (in)
PREC	Mean annual precipitation (inches)
q	Excess rainfall (in)
q _p	Peak flow (cfs)
q _t	Discharge at time t
Q	Peak runoff (cfs)

LIST OF PARAMETERS (cont'd)

Q'	Total volume of triangular unit hydrograph (in)
Q_t	Runoff at time t (cfs)
Q'_t	Accumulated Volume at time t
Q_{Tr}	Flow rate for corresponding return period
RC	Soil runoff coefficient
$R(t)$	Accumulated runoff (in)
s	Slope (ft/ft or m/m)
s'	Average surface slope (%)
s_y	Standard deviation
S	Ultimate abstraction
SL	Slope (feet per mile)
STOR	Storage (%)
t	Time (hrs)
t_b	Time (duration) of base
t_c	Time of concentration (min)
t_p	Time to peak
t_r	Recession duration
t_t	Travel time (min)
T_r	Return period
u	Ordinate of the unit hydrograph (cfs/in)
v	Velocity
V	Volume
$V(C_s)$	Variance of the station skew
$V(C_m)$	Variance of the map skew
V_{UH}	Volume of the unit hydrograph
V_t	Total volume of the triangular unit hydrograph
W	Weight
x_i	Annual maximum flow values
\bar{y}	Mean
y_i	Logarithmic annual maximum flow values
y_H	Logarithm of a high outlier
y_L	Logarithm of a low outlier

Chapter 3 - RUNOFF AND ITS ESTIMATION

3.1 - FACTORS AFFECTING RUNOFF

In the discussion throughout Chapter 2, it was noted that *only a part of* the rainfall is converted to surface runoff. Figure 3.1.1 presents a comprehensive view of one of the many possible interactions between rainfall and the earth as time increases from the beginning of rainfall.

The horizontal axis in Figure 3.1.1 represents the time from the start of the rainfall and the vertical axis represents a corresponding depth. This particular diagram represents an extensive storm of uniform intensity on a dry basin.

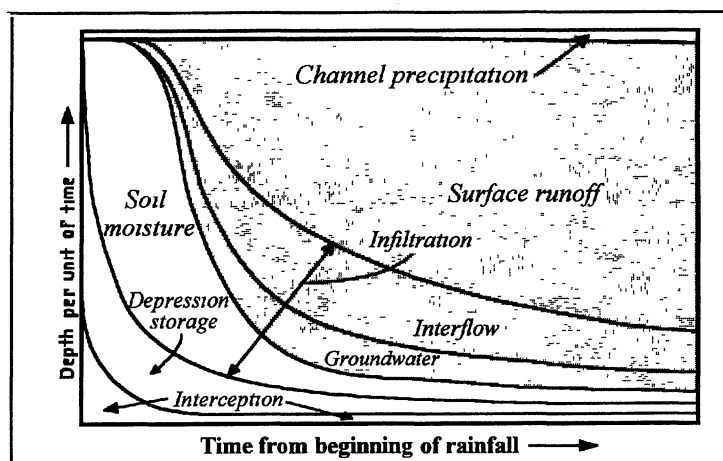


Figure 3.1.1
Schematic Diagram of the Disposition of Storm Rainfall
(after Linsley et al., 1975)

The shaded portion of the diagram represents the portion of the rainfall which will become flow measured at the point under consideration. During the early period of the storm, *channel precipitation*, rainfall that falls directly on the channel, is the only input to flow. As the storm progresses, other factors dominate: depression storage; interception; groundwater flow; interflow; infiltration; and surface runoff.

Depression storage is the volume of water which collects in natural depressions or ponds on impermeable surfaces. Once the rainfall intensity exceeds the local infiltration capacity of

the soil, surface depressions, natural and man-made, begin to fill. After smaller depressions are filled, overland flow begins which in turn fills larger depressions or flows directly to the channel.

Interception is rainfall which is held in storage by vegetation.

Infiltration is the seepage of rainfall into the subsurface. The infiltration capacity of the soil depends upon the soil type, moisture content of the soil, amount of organic matter present, vegetal cover, season of the year and rainfall intensity. As a storm progresses, infiltration rate decreases because the capillary spaces in the soil are filled. Infiltration is affected by urbanization as permeable soils are replaced with impermeable structures. Obviously, infiltration opportunity decreases and runoff increases.

The contribution of *groundwater* to channel flow does not fluctuate rapidly because of long flowpaths and low velocities through the soil. The groundwater contributes to the channel if the water table intersects the channel.

Interflow is the portion of water which infiltrates the soil surface and moves laterally through the upper layers of the soil until it re-emerges or enters the channel. It is dependent upon the soil type and the geology of the watershed under consideration. For some watersheds, subsurface flow may be the dominant contribution to stormwater runoff.

The last portion of Figure 3.1.1 is the *surface runoff* which starts at zero and increases as the storm progresses. As the storm progresses, runoff levels off to a relatively constant percentage of rainfall. In urban areas with a high percentage of impervious area and low levels of detention storage, the major contributor to flow in the channel is surface runoff.

3.2 - THE RATIONAL METHOD

The rational method is one of the oldest, simplest, and most widely used and often criticized methods employed in the determination of peak discharges from a given watershed. It was first introduced into this country by Kuichling in 1889, and a survey indicated that it is used in 90 percent of the engineering offices in the United States (Ardis, et al., 1969). This popularity can probably be attributed to its simplicity, "ease" of application and tradition.

The fundamental idea behind the rational method is that the peak rate of surface outflow from a given watershed is proportional to the watershed area and average rainfall intensity over a period of time just sufficient for all parts of the watershed to contribute to the outflow. The constant of proportionality reflects the characteristics of the watershed, such as imperviousness, which affect runoff. The rational formula is written as:

$$Q = C i A \quad (3.2.1)$$

where Q is the peak runoff (cubic feet per second - cfs), C is the ratio of peak runoff rate to average rainfall rate over the watershed during the time of concentration (runoff coefficient), i is the rainfall intensity (inches/hour), and A is the contributing area of watershed under consideration (acres).

It should be noted that the conversion from acres-inches/hour to cfs is 1.008. This value is usually neglected and it is for these units that the formula was termed "rational".

For metric units,

$$Q = 0.02778 C i A \quad (3.2.2)$$

where Q is the peak discharge (cubic meters per second - m³/s), C is the ratio of peak runoff rate to average rainfall rate over the watershed during the time of concentration (runoff coefficient), i is the rainfall intensity (centimeters/hour - cm/hr), and A is the contributing area of watershed under consideration (hectares - ha).

The coefficient 0.02778 arises from the conversion from hectare-centimeters/hour to m³/s.

In general, the rational method should be applied to drainage basins less than 200 acres (81 ha) in area and is best suited for well-defined drainage basins. Some local ordinances limit the use of the rational method to basins to areas much smaller than the 200 acres (81 ha). Application of the rational method is illustrated in Example 3.2.1.

The basic assumptions used in the application of the rational formula are as follows.

1. The return period of the peak discharge is the same as that of the rainfall intensity.
2. The rainfall is uniform in space over the watershed under consideration.
3. The storm duration associated with the peak discharge is equal to the time of concentration for the drainage area (the time for the most hydraulically-distant point to contribute to the peak outflow at the point under consideration).
4. The runoff coefficient C is not influenced by the return period.
5. The runoff coefficient C is independent of the storm duration for a given watershed and reflects any changes in infiltration rates, soil types and antecedent moisture conditions.

3.2.1 - Determination of Runoff Coefficient, C

Values of the runoff coefficient are given in Table 3.2.1 for rural areas and Table 3.2.2 for urban areas. Table 3.2.2 presents runoff coefficients for particular types of urban areas and Table 3.2.3 gives coefficients which are used to compute a weighted C based on the actual percentage of lawns, streets, roofs, etc. The determination of the runoff coefficient is illustrated in Example 3.2.1.

Table 3.2.1
Rural Runoff Coefficients (Schwab et al., 1966)

Vegetation and <u>Topography</u>	Open Sandy <u>Loam</u>	<u>Soil Texture</u>	
		Clay and Silt <u>Loam</u>	Tight Clay
Woodland			
Flat 0-5% slope	0.10	0.30	0.40
Rolling 5-10% slope	0.25	0.35	0.50
Hilly 10-30% slope	0.30	0.50	0.60
Pasture			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
Cultivated			
Flat	0.30	0.50	0.60
Rolling	0.40	0.60	0.70
Hilly	0.52	0.72	0.82

As mentioned before, this coefficient represents the runoff-rainfall ratio and includes many factors such as type of cover, soil types, infiltration, evaporation, evapo-transpiration, and any antecedent moisture condition. For many years it has been known that C actually does not remain constant during a storm (Horner, 1910). The strong dependence on "engineering judgment" in selecting a runoff coefficient is one of the main weaknesses of the rational method.

Table 3.2.2
Urban Runoff Coefficients for the Rational Method (ASCE, 1992)

<u>Description of Area</u>	<u>Runoff Coefficients</u>
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

Table 3.2.3
Values Used to Determine a Composite Runoff Coefficient for an Urban Area
(ASCE, 1992)

<u>Character of Surface</u>	<u>Runoff Coefficients</u>
Pavement	
Asphalt and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.05 to 0.10
Average, 2 to 7 percent	0.10 to 0.15
Steep, 7 percent	0.15 to 0.20
Lawns, heavy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35
Water Impoundment	1.00

3.2.2 - Determination of a Time of Concentration

The rational method assumes that the storm duration is equal to the time of concentration, which is the travel time for the most hydraulically distant point to contribute to the point under consideration. The time of concentration (t_c) is calculated in order to estimate the intensity of the design storm.

The travel time to a given location is the sum of the overland flow time, the gutter flow time, and the sewer flow time. The time of concentration at this location is the longest travel time when all paths are considered. For sewer design, this means that the inlet time for each sub-basin must be compared to the travel times from all upstream sub-basins and the longest time selected as the local time of concentration.

Table 3.2.4 presents some commonly used formulae to determine the overland flow time. Most of these equations relate this time to the basin length, slope, and surface roughness. Two equations, by Izzard and Ragan, include rainfall intensity and hence, an iterative solution is necessary to compute t_c . The equation used for the computation of t_c should be chosen according to the specific site conditions, following the guidelines applicable to those equations. Gutter flow times can be estimated using the Manning formula which is discussed in Chapter 5. Often the inlet time is merely assumed. Commonly used values for the inlet time vary from 5 to 20 minutes (ASCE, 1986). The SCS (1986) has adopted the simple curves presented in Figure 3.4.5 for computing the time of concentration in urban areas.

These inlet times are added to the flow time in the pipe or channel to determine the travel time at the next point of interest. However, if the inlet time for this point is greater than the upstream travel time, the inlet time is used as t_c in the subsequent calculation.

The sewer flow time is usually calculated by choosing a pipe or channel configuration and calculating the velocity. The travel time is then found by:

$$t_t = \frac{L}{v} \frac{1 \text{ minute}}{60 \text{ seconds}} = \frac{L A'}{Q} \frac{1 \text{ minute}}{60 \text{ seconds}} \quad (3.2.3)$$

where t_t is the travel time in pipe (min), L is the reach length (ft, m), v is the velocity in reach (ft/sec, m/sec) = Q/A' , and A' is the cross-section area (ft², m²).

Table 3.2.4
Equations for Determining Overland Flow Time

GENERAL NOTE ON t_c FORMULAE: t_c will have units of minutes for all following equations.

Federal Aviation Administration (1970)

$$t_c = \frac{K (1.1 - C) L^{0.5}}{S'^{\frac{1}{3}}}$$

where K is equal to 1.8 for U.S customary units (3.26 for metric units), C is the rational method runoff coefficient, L is the length of overland flow in ft (m), and s' is the average surface slope, %.

This method was developed from airfield drainage data assembled by the Corps of Engineers. The method is intended for use on airfield drainage areas, but has been used for overland flow in urban areas (Chow et al.,1988).

Kinematic Wave (Ragan, 1972)

$$t_c = \frac{K (L^{0.6} n^{0.6})}{i^{0.4} S^{0.3}}$$

where K is equal to 0.94 for U.S. customary units (2.78 for metric units), L is the length of overland flow in ft (m), n is Manning's roughness coefficient for sheet flow (tabulated in Table 3.2.6), i is the rainfall intensity, in/hr (cm/hr), and s is the average slope of overland flow path, ft/ft (m/m).

The length of the overland flow segment generally should be limited to 300 feet (90 m). This method requires iteration since both i and t_c are unknown. The steps for iteration are:

1. Assume a trial value of rainfall intensity (i)
2. Find the overland travel time (t_c)
3. Find the actual rainfall intensity for a storm duration of t_c from the appropriate intensity-duration-frequency (IDF) curve for the area under consideration.
4. Compare rainfall intensities, if they are not the same, select a new trial rainfall and repeat step 1 (Wanielista, 1990).

Table 3.2.4 (cont'd)

Kerby (1959)

$$t_c = K (L N s^{-0.5})^{0.467}$$

where K is equal to 0.83 (US Customary units) or 1.44 (Metric units), L is the length of flow in ft (m), s is the average slope of overland flow, ft/ft (m/m), and N is the retardance roughness coefficient given in Table 3.2.5.

The length used in the equation is the straight-line distance from the most distant point of the watershed to the outlet, measured parallel to the slope of the land until a well-defined channel is reached. Watersheds of less than 10 acres were used to calibrate the model; slopes were less than 1%; N values were 0.8 and less and surface flow dominated (McCuen, 1989).

Izzard (1946)

$$t_c = \frac{K(Bi + c') L^{\frac{1}{3}}}{s^{\frac{1}{3}} i^{\frac{2}{3}}}$$

where K is equal to 41.025 for U.S. customary units (113.391 for metric), B is equal to 0.0007 for U.S. customary units (0.00027 for metric), c' is the retardance coefficient given in Table 3.2.7, i is the rainfall intensity, in/hr (cm/hr), L is the length of flow path in ft (m), and s is the slope of overland flow path, ft/ft (m/m).

The product of i and L must be less than 500 in-ft/hr (390 cm-m/hr) to consider using this formula. In addition, well defined channels should **not** be present. This method was developed in laboratory experiments for the overland flow on roadway and turf surfaces.

Table 3.2.5
Values of N for Kerby's Formula (Kerby, 1959)

<u>Type of Surface</u>	<u>N</u>
Smooth impervious surface	0.02
Smooth bare packed soil	0.10
Poor grass, cultivated row crops or moderately rough bare surface	0.20
Deciduous timberland	0.60
Pasture or Overage grass	0.40
Conifer timberland, deciduous timberland with deep forest litter or dense grass	0.80

Table 3.2.6
Manning's *n* Roughness Coefficients for Sheet Flow (Engman, 1983)

<u>Type of Surface</u>	<u><i>n</i></u>
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (with no residue)	0.05
Cultivated soils	
Cover ≤ 20%	0.06
Cover ≥ 20%	0.17
Grass	
Short grass, prairie	0.15
Dense grass	0.24
Bermudagrass	0.41
Range	0.13
Woods	
Light underbrush	0.40
Dense underbrush	0.80

Table 3.2.7
Values of *c* for Izzard's Formula

<u>Surface</u>	<u><i>c</i></u>
Smooth asphalt surface	0.007
Concrete pavement	0.012
Tar and gravel pavement	0.017
Closely clipped sod	0.046
Dense bluegrass turf	0.060

3.2.3 - Application of the Rational Method

The following procedure is used to apply the Rational Method.

Step 1: Determine the contributing basin area *A* (acres or hectares) by using USGS topographical maps, Indiana county drainage maps, maps developed from a survey of the area, or plans made specifically for the basin. This area is found by using a planimeter or digitizer.

Step 2: By the use of Table 3.2.1 for rural areas or Table 3.2.2 for urban areas, estimate the appropriate value of *C*. If the land use is mixed, a composite *C* value is estimated from Table 3.2.3 or is determined by:

$$C_{comp} = \frac{(C_1 A_1 + C_2 A_2 + \dots + C_n A_n)}{A_t} \quad (3.2.4)$$

where C_1, C_2, \dots, C_n are the runoff coefficients (taken from Tables 3.2.1 and 3.2.2) associated with component areas A_1, A_2, \dots, A_n and $A_t = A_1 + A_2 + \dots + A_n$.

Step 3: Determine the time of concentration. The overland flow time of concentration for the basin can be determined by using one of the equations listed in Table 3.2.4 or the curves shown on Figure 3.4.5. The travel time in rills and channels are added to the overland flow travel times.

Step 4: Determine the intensity and peak discharge. Enter an intensity-duration-frequency curve with the storm duration equal to the time of concentration and, by choosing a curve corresponding to the appropriate return period, read off the intensity value. This is then multiplied by the area and runoff coefficient to determine the peak discharge rate.

If there is another basin downstream, the first time of concentration is added to the travel time in the channel or pipe using Equation 3.3. This is then compared to the inlet time of the second basin and the larger of two is used as the new time of concentration for the combined areas.

Example 3.2.1

This example problem illustrates the use of the rational method

A watershed in South Bend is composed of three separate subbasins, Basin 1, Basin 2, and Basin 3, as shown in the Figure 3.2.1. Runoff from the uppermost basin, Basin 1, flows to point "a", where all flow from the subbasin is collected at an inlet. A pipe runs from point "a" to point "b", where another inlet collects the additional runoff from Basin 2. A second pipe runs from point "b" to point "c", where the additional runoff from Basin 3 is collected. If the basins have the following characteristics, calculate the runoff at points "a", "b", and "c" by the rational method for a 10-year return period. Assume the pipes flow with a velocity of 3 ft/sec. Pipe "a-b" has a length of 600 ft. Pipe "b-c" has a length of 90 ft.

<i>Basin</i>	<i>Area (acres)</i>	<i>Basin Travel Time (min)</i>	<i>Land Use</i>
Basin 1	24	10	Apartment Complex
Basin 2	13.7	37	Park
Basin 3	3.8	31	Light Industry

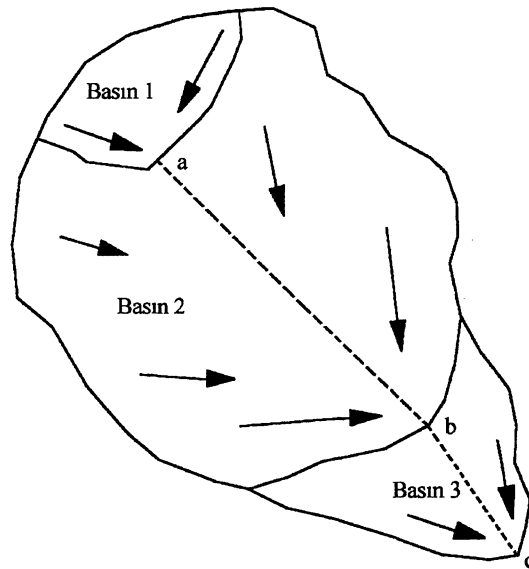


Figure 3.2.1
Hypothetical Watershed for Example 3.2.1

Step 1: The areas of each subbasin are given in the above table.

Step 2: Calculate the runoff coefficients. Obtain the runoff coefficients from Tables 3.2.1 - 3.2.2. Basin 1 (Apartment): $C_1 = 0.60$, Basin 2 (Parks, cemeteries): $C_2 = 0.15$, Basin 3 (Industrial, light): $C_3 = 0.65$.

Using Eq. 3.2.4, compute composite C values at points "a", "b" and "c":

Point "a": $C_{\text{comp}} = (0.60 \cdot 2.4) / 2.4 = 0.60$

Point "b": $C_{\text{comp}} = (0.60 \cdot 2.4 + 0.15 \cdot 13.7) / (2.4 + 13.7) = 0.22$

Point "c": $C_{\text{comp}} = (0.60 \cdot 2.4 + 0.15 \cdot 13.7 + 0.65 \cdot 3.8) / (2.4 + 13.7 + 3.8) = 0.30$

Step 3: Calculate the time of concentration for points "a", "b", and "c".

Point "a": $t_c = 10 \text{ min}$

Point "b": Compare (Basin 1 travel time + pipe flow time from "a" to "b") to Basin 2 travel time.

Using Eq. 3.2.3, pipe flow time, t , from "a" to "b" is $600/3 \cdot 1/60 = 3.33$ minutes.

Basin 1 travel time + pipe flow time_{a-b} = $10 + 3.33 = 13.33$ minutes

Basin 2 travel time = 37 minutes $t_c = 37 \text{ minutes}$

Point "c" : Compare (the time of concentration at point "b" + pipe flow time from "b" to "c") to Basin 3 travel time.

Using Eq. 3.2.3, pipe flow time from "b" to "c" is $90/3 * 1/60 = 0.5$ minutes.

t_c at "b" + pipe flow time_{b-c} = $37 + 0.5 = 37.5$ minutes

Basin 3 travel time = 31 minutes $t_c = 37.5$ minutes

Step 4: Calculate the rainfall intensities for points "a", "b", and "c". Recall that the time of the storm duration will be set equal to the t_c for each subbasin. Since the t_c is measured in minutes, it must be converted to hours.

Using Equation 2.2.13

$$i = \frac{c T_r^\alpha}{(t + d)^\beta}$$

Where : $c=1.7204$ $\alpha=0.1753$ $d=0.485$ $\beta=1.6806$,
From Table 2.2.2 (South Bend for $5 \text{ min} < t \leq 60 \text{ min}$)

Point "a" :

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{1.7204 \cdot (10)^{0.1753}}{(\frac{10}{60} + 0.485)^{1.6806}} = 5.29 \text{ inches/hr}$$

Point "b" :

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{1.7204 * 10^{0.1753}}{(\frac{37}{60} + 0.485)^{1.6806}} = 2.19 \text{ inches/hr}$$

Point "c" :

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{1.7204 * 10^{0.1753}}{(\frac{37.5}{60} + 0.485)^{1.6806}} = 2.16 \text{ inches/hr}$$

Step 5: Calculate the flowrates at points "a", "b", and "c":

Point "a" : $Q = C i A = 0.60 * 5.29 * 2.4 = 7.62 \text{ cfs}$

Point "b" : $Q = C i A = 0.22 * 2.19 * (2.4 + 13.7) = 7.76 \text{ cfs}$

Point "c" : $Q = C i A = 0.30 * 2.16 * (2.4 + 13.7 + 3.8) = 12.90 \text{ cfs}$

3.3 - SOIL CONSERVATION SERVICE CURVE NUMBER METHOD

The Soil Conservation Service (SCS) Curve Number (CN) Method was developed to determine the quantity of runoff from a given amount of precipitation (NEH-4, 1972). The CN method uses basin soil and cover types, rainfall depth, and the antecedent moisture condition to predict the runoff volume. The method is relatively easy to use and has wide applicability. It has been recommended for both rural and urban watersheds. Although there are no limits on the watershed area to which it can be applied, the area must be small enough so that the rainfall intensity is uniform. If the soil type varies within the basin, the basin should be divided into the sub-basins of similar character or a composite CN should be calculated.

3.3.1 - Theory of the Curve Number Method

The basic principle of the curve number method can be described in conjunction with Figure 3.3.1, which is a graph of quantity versus time. The upper line represents the total rainfall $P(t)$ as a function of time. This is assumed to be a linear function for convenience. The dashed line concave to the horizontal axis represents the total abstraction or loss, $L(t)$, as a function of time. The total abstraction is the sum of the initial abstraction, I_a , due mainly to surface wetting, depression storage, and the total infiltration, $F(t)$.

Figure 3.3.1 is a diagram of accumulated rainfall, runoff and infiltration occurring during the storm being studied. It is assumed that no runoff occurs until I_a has been satisfied and that after a sufficiently long period of time, $F(t)$ reaches a constant saturation value denoted by $S - I_a$. The symbol, S , stands for the ultimate total abstraction and depends on soil type, cover, and antecedent moisture condition.

The cumulative runoff, $R(t)$, is the difference between the total rainfall and the total abstraction. The accumulated runoff is determined by Equation 3.3.1.

$$R(t) = \frac{(P(t) - 0.2S)^2}{P(t) + 0.8S} \quad (3.3.1)$$

Where $P(t)$ is the cumulative rainfall (inches)

In Equation 3.3.1, if $P(t) \leq 0.2S$, then $R(t)$ is equal to zero.

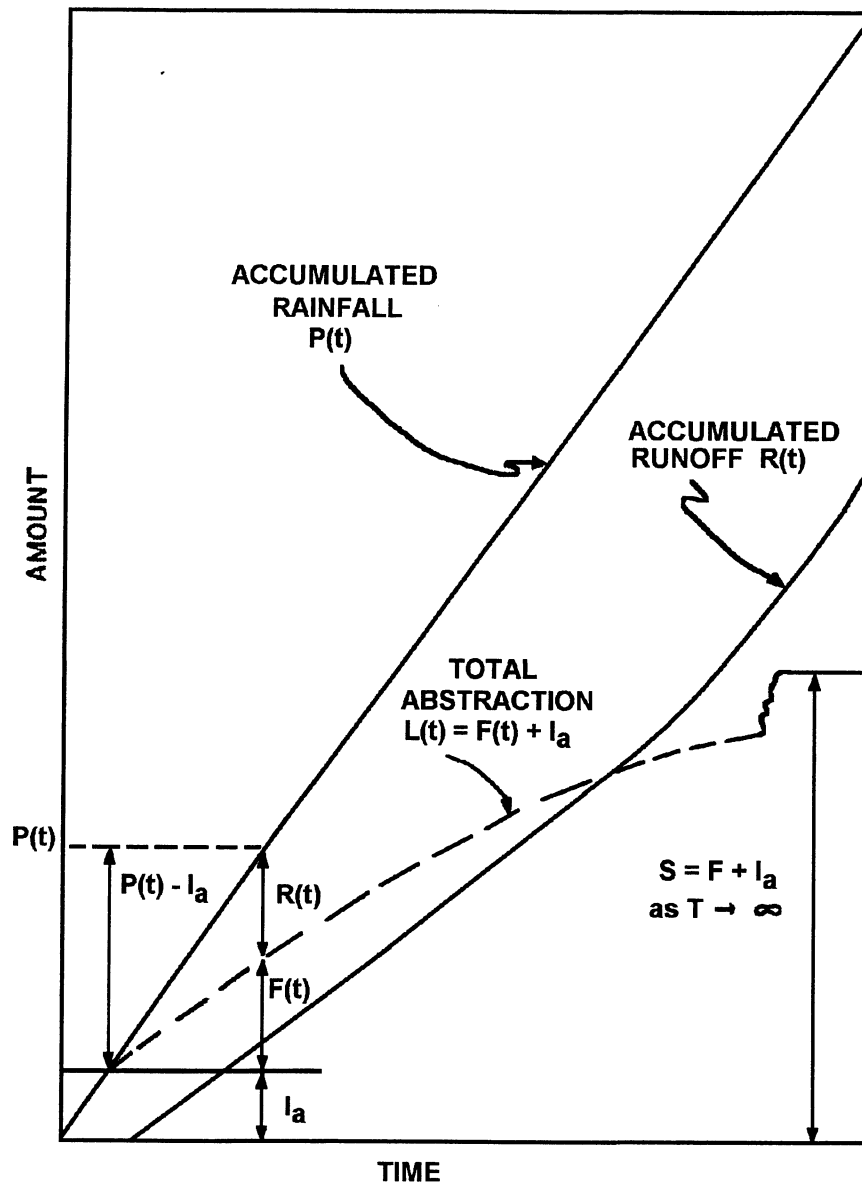


Figure 3.3.1
Diagram of Accumulated Rainfall, Runoff and Infiltration (SCS; NEH-4)

3.3.2 - Determination of the Parameter S

In the determination of the ultimate abstraction S, the soil characteristics are considered. Soils have been classified by the SCS into four groups based on infiltration capacity. The criteria for each soil group are described in Table 3.3.1. Soils common in the United States have been analyzed and placed in the appropriate category based on Table 3.3.1. Soils common to Indiana are given in Table 3.3.2. Soils at a particular site can be determined by using county soil maps distributed by the regional Soil Conservation Service office.

The antecedent moisture condition, AMC, which is an indication of the wetness of the basin under consideration, used to be a factor in the design. However, the latest SCS publications now refer to an average runoff condition, ARC, which is AMC II.

Table 3.3.1
Criteria Used by the Soil Conservation Service in the
Classification of Soils - National Engineering Handbook
Hydrology: Section 4 (SCS; NEH-4)

<u>Soil Group</u>	<u>Characteristics</u>
A	Soils in this category have a high infiltration rate even when thoroughly wetted and consist mainly of deep, well-to excessively-drained sands or gravels. (Lowest runoff potential).
B	Soils in this category have moderate infiltration rates when thoroughly wetted and consist of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures.
C	Soils in this category have slow infiltration rates when thoroughly wetted and consist mainly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine textures.
D	Soils in this category have a very slow infiltration rate then thoroughly wetted and consist mainly of clay soils with high swelling potential, soils with a permanently high water table, soils with a clay or clay layer at or near the surface, and shallow water over nearly impervious material. (Highest runoff potential).

Once the soil classification is known, a curve number CN can be found given the type of cover in the watershed. The relationship between S and the curve number is:

$$S = \left(\frac{1000}{CN} \right) - 10 \quad (3.3.2)$$

Table 3.3.2 and Table 3.3.3 may be used to determine the appropriate CN for the particular cover and soil type in the watershed. Higher curve numbers occur with higher impervious covers and with soils with slower infiltration rates. These numbers, when used with Equation 3.3.2 are consistent with the physical process. If a CN of 100 is used for an ideal impervious cover, the runoff predicated by Equation 3.3.1 is equal to the rainfall. The CN values presented in Tables 3.3.2 and 3.3.3 are for the ARC (AMC II).

3.3.3 - Application of the CN Method

Once a CN has been determined, a value of S may be calculated by using Equation 3.3.2. By selecting the rainfall depth for a particular storm, Equation 3.3.1 may be solved for runoff R(t). An alternative to using Equation 3.3.1 is the graphical solution presented in Figure 3.3.2. The units of R(t) are inches. By multiplying R by the basin area and converting units, volume of runoff may be determined for the watershed under consideration.

For a basin which has different soil types and/or land uses, a composite or weighted CN may be determined by Equation 3.3.3.

$$CN_{comp} = \frac{(CN_1 A_1 + CN_2 A_2 + \dots + CN_n A_n)}{A_t} \quad (3.3.3)$$

Where $CN_1, CN_2 \dots CN_n$ are the curve numbers associated with component areas $A_1, A_2, \dots A_n$ and $A_t = A_1 + A_2 + \dots + A_n$.

Table 3 3 2
Hydrologic Soil Groups For Indiana Soils
(Shown in parentheses after the soil classification)

Abscota (A)	Birkbeck (B)	Clyde (B/D)	Ebal (B)	Gudgel (C)	Kibbie (B)
Ackerman (A/D)	Bloomfield (A)	Cobbsford (D)	Eden (C)	Guthrie (D)	Kings (D)
Ade (A)	Blount (C)	Coesse (C/D)	Edenton (C)		Kokomo (B/D)
Adrian (A/D)	Bobtown (C)	Cohoctah (B/D)	Edwards (B/D)		Kosciusko (B)
Alford (B)	Bonnell (C)	Coloma (A)	Eel (B)	Hagerstown (C)	Kurtz (C)
Algansee (B)	Bonnie (C/D)	Colyer (D)	Eldean (B)	Haney (B)	
Algiers (C/D)	Bono (D)	Comfrey (B/D)	Elkinsville (B)	Hanna (B)	
Alida (B)	Booker (D)	Conotton (B)	Elliott (C)	Harpster (B/D)	Lafayette (B)
Allison (B)	Boonesboro (B)	Conover (C)	Elston (B)	Haskins (C)	La Hogue (B)
Alvin (B)	Boots (A/D)	Conrad (A/D)	Evansville (B/D)	Haubstadt (C)	Landes (B)
Ambraw (B/D)	Bourbon (B)	Coolville (C)		Haymond (B)	Lash (B)
Andres (B)	Bowes (B)	Corwin (B)		Hennepin (B)	Laramie (B)
Aptakisic (B)	Boyer (B)	Cory (C)	Fabius (B)	Henshaw (C)	Lawrence (C)
Armiesburg (B)	Brady (B)	Corydon (D)	Fairmount (D)	Hickory (C)	Lenawee (B/D)
Ashkum (B/D)	Branch (B)	Coupee (B)	Fairpoint (C)	High Gap (C)	Lindsie (C)
Atkins (D)	Brems (A)	Craigmile (B/D)	Farmington (C)	Hillsdale (B)	Linkville (C)
Aubbeenaubee (B)	Brenton (B)	Crane (B)	Faxon (B/D)	Holton (C)	Linwood (A/D)
	Bromer (C)	Crawleyville (B)	Fincastle (C)	Homer (B)	Lisbon (B)
Ava (C)	Bronson (B)	Crider (B)	Flanagan (B)	Hononegah (A)	Lobdell (B)
Avonburg (D)	Brookston (B/D)	Crosby (C)	Foresman (B)	Hoopeston (B)	Lomax (B)
Ayr (B)	Bruno (A)	Crosier (C)	Fox (B)	Hoosierville (C)	Longlois (B)
Ayrmount (B)	Bryce (D)	Cuba (B)	Frederick (B)	Hosmer (C)	Losantville (C)
Ayrshire (C)	Burside (B)	Cyclone (B/D)	Free (B/D)	Houghton (A/D)	Lucas (D)
			Fulton (D)	Hoytville (C/D)	Lydick (B)
				Huntington (B)	Lyles (B/D)
				Huntsville (B)	
Banlic (C)	Cadiz (B)	Dana (B)			
Barce (B)	Camden (B)	Darroch (B)	Genesee (B)		
Barry (B/D)	Cana (C)	Darroch, bedrock	Gessie (B)		Mahalasville
Bartle (D)	Caneyville (C)	substratum	Gilboa (B)	Iona (B)	(B/D)
Battleground (B)	Carlisle (A/D)	(C)	Gilford (B/D)	Ipava (B)	Maplehill (C)
Baxter (B)	Carmel (C)	Dearborn (B)		Iroquois (B/D)	Markel (B)
Beanblossom (B)	Carmi (B)	Del Rey (C)		Iva (C)	Markham (C)
Beasley (C)	Casco (B)	Deputy (C)	Gilford, stratified		Markland (C)
Beaucoup (B/D)	Catlin (B)	Derinda (C)	substratum		Markton (C)
Beckville (B)	Celina (C)	Desker (A)	(D)	Jasper (B)	Martinsville (B)
Bedford (C)	Ceresco (B)	Dickinson (B)	Gilpin (C)	Jennings (C)	Martisco (B/D)
Beecher (C)	Chagrin (B)	Digby (B)	Ginat (D)	Johnsburg (D)	Maumee (A/D)
Belknap (C)	Chalmers (B/D)	Door (B)	Glenhall (B)	Jules (B)	McGary (C)
Belleville (B/D)	Chatterton (A)	Dowagiac (B)	Glynwood (C)	Junius (C)	Medora (B)
Belmore (B)	Cheektowaga (D)	Driftwood (C/D)	Gosport (C)		Medway (B)
Berks (C)	Chelsea (A)	Drummer (B/D)	Granby (A/D)		Mellott (B)
Bethesda (C)	Chetwynd (B)	Du Page (B)	Gravelton (B/D)	Kalamazoo (B)	Mermill (B/D)
Bewleyville (B)	Cincinnati (C)	Dubois (C)	Grayford (B)	Kendallville (B)	Metamora (B)
Billett (B)	Clarence (D)	Dunning (D)	Griswold (B)	Kentland (A/D)	Meta (B)
Birds (C/D)	Cleremont (D)		Grovecity (B)	Kerston (A/D)	Miami (B)

Notes 1 Two hydrologic soil groups shown together, such as B/C, indicate the drained/undrained conditions
2 Any modifiers shown, e g , bedrock substratum, refer to a specific soil series phase found in the soil map legend

Table 3 3 2
Hydrologic Soil Groups For Indiana Soils
(Shown in parentheses after the soil classification)

Miamian (C)	Owosso (B)	Rensselaer,	Sloan (B/D)		Xenia (B)
Middlebury (B)		nonstratified	Sparta (A)	Uniontown (B)	
Milford (B/D)		substratum	Spinks (A)		Zaborosky (B)
Millbrook (B)	Palms (A/D)	(C)	St Charles (B)		Zadog (A/D)
Millgrove (B/D)	Papineau (C)	Richardsville (B)	St Clair (D)	Varna (C)	Zanesville (C)
Millsdale (B/D)	Parke (B)	Riddles (B)	Starks (C)	Vigo (D)	Zipp (D)
Milton (C)	Parr (B)	Ridgeville (B)	Steff (C)	Vincennes (C/D)	
Monitor (C)	Pate (C)	Rimer (C)	Stendal (C)	Volinia (B)	
Montgomery (D)	Patton (B/D)	Riverdale (A)	Stockland (B)		
Montmorenci (B)	Pekin (C)	Roby (C)	Stonelick (B)		
Morley (C)	Pella (B/D)	Rockcastle (D)	Stoy (C)	Wabash (D)	
Morocco (B)	Peoga (C)	Rockfield (B)	Strawn (B)	Wakeland (C)	
Moundhaven (A)	Peotone (B/D)	Rockton (B)	Strole (C)	Wallkill (C/D)	
Muldavia (B)	Petrolia (B/D)	Rodman (A)	Suman (B/D)	Warners (C/D)	
Mulvey (B)	Pewamo (C/D)	Ross (B)	Sumava (B)	Warsaw (B)	
Mundelein (B)	Philo (B)	Rossmoyne (C)	Sunbury (B)	Wasepi (B)	
Muren (B)	Piankeshaw (B)	Ruark (B/D)	Swanwick (D)	Washtenaw	
Muskego (A/D)	Pike (B)	Rush (B)	Switzerland (B)	(C/D)	
Muskingum (C)	Pineville (B)	Russell (B)	Swygert (C)	Watseka (B)	
Mussey (B/D)	Pinhook (B/D)	Ryker (B)	Sylvan (B)	Waupecan (B)	
	Piopolis (C/D)		Symerton (B)	Wauseon (B/D)	
	Pipestone (B)			Wawasee (B)	
Napoleon (A/D)	Plainfield (A)	Sable (B/D)		Waynetown (C)	
Nappanee (D)	Plano (B)	Saranac (C/D)	Taftown (B)	Wea (B)	
Negley (B)	Pope (B)		Taggart (C)	Weikert (C/D)	
Nesius (A)	Princeton (A)	Saranac, gravelly	Tama (B)	Weinbach (C)	
Newark (C)	Prochaska (A/D)	substratum	Tawas (A/D)	Weisburg (C)	
Newglarus (B)	Proctor (B)	(C)	Tecumseh (B)	Wellston (B)	
Newton (A/D)		Saugatuck (C)	Tedrow (B)	Westland (B/D)	
Nicholson (C)		Sawabash (B)	Thackery (B)	Wheeling (B)	
Nineveh (B)	Quinn (B/D)	Sciotoville (C)	Throckmorton	Whitaker (C)	
Nolin (B)		Seaford (B)	(B)	Whitson (D)	
		Sebewa (B/D)	Tice (B)	Wilbur (B)	
	Ragsdale (B/D)	Selfridge (B)	Tilsit (C)	Wilhite (C/D)	
Oakville (A)	Ragsdale,	Selma (B/D)	Tippecanoe (B)	Willette (A/D)	
Ockley (B)	overwash (B)	Seward (B)	Toledo (D)	Williamsport (C)	
Octagon (B)	Rahm (C)	Shadeland (C)	Toronto (C)	Williamstown	
Odell (B)	Rainsville (B)	Shakamak (C)	Toto (B/D)	(C)	
Oldenburg (B)	Randolph (C)	Shipshe (B)	Tracy (B)	Wingate (B)	
Onarga (B)	Rarden (C)	Shoals (C)	Trappist (C)	Wirt (B)	
Ormas (B)	Raub (C)	Sidell (B)	Treaty (B/D)	Wolcott (B/D)	
Orrville (C)	Rawson (B)	Simonin (B)	Trevlac (B)	Woodmere (B)	
Oshtemo (B)	Reddick (B/D)	Sisson (B)	Troxel (B)	Woolper (C)	
Otwell (C)	Reesville (C)	Skelton (B)	Tuscola (B)	Wynn (B)	
Ouateno (A)	Rensselaer (B/D)	Sleeth (C)	Tyner (A)		

Notes 1 Two hydrologic soil groups shown together, such as B/C, indicate the drained/undrained conditions
2 Any modifiers shown, e g , bedrock substratum, refer to a specific soil series phase found in the soil map legend

Table 3.3.3
Runoff Curve Numbers for Urban Areas (SCS, 1986)

Cover Type and Hydrologic Condition	Curve Numbers for Hydrologic Soil Group			
	A	B	C	D
<i>Undeveloped Areas</i>				
Cultivated Land				
Without conservation treatment	72	81	88	91
With conservation treatment	62	71	78	81
Pasture or range land				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow				
Good condition	30	58	71	78
Wood or forest land				
Thin stand, poor cover, no mulch	45	66	77	83
Good cover	25	55	70	77
<i>Fully developed urban areas (with established vegetation)</i>				
Open space (lawns, parks, golf courses, cemeteries)				
Poor condition (grass cover < 50%)	68	79	86	89
Fair condition (grass cover 50% to 75%)	49	69	79	84
Good condition (grass cover > 75%)	39	61	74	80
Impervious areas:				
Paved parking lots, roofs, driveways, etc (excluding right-of-way)	98	98	98	98
Streets and roads				
Paved curb and storm sewers (excluding right-of-way)	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89
Urban Districts				
Commercial and business (85% impervious)	89	92	94	95
Industrial (72% impervious)	81	88	91	93
Residential				
1/8 acre or less, townhouses (65% impervious)	77	85	90	92
1/4 acre (38% impervious)	61	75	83	87
1/3 acre (30% impervious)	57	72	81	86
1/2 acre (25% impervious)	54	70	80	85
1 acre (20% impervious)	51	68	79	84
2 acre	46	65	77	82
<i>Developing Urban Areas</i>				
Newly graded areas (no vegetation)	77	86	91	94

Table 3.3.4
Runoff Curve Numbers for Agricultural Lands (SCS, 1986)

Cover Type and Hydrologic Condition	Curve Numbers for Hydrologic Soil Group			
	A	B	C	D
Pasture, grassland, or range with continuous forage for grazing				
Poor	68	79	86	89
Fair	49	69	79	84
Good	39	61	74	80
Meadow with continuous grass, protected from grazing and generally mowed for hay	30	58	71	78
Brush/brush-weed-grass mixture with brush being the major element				
Poor	48	67	77	83
Fair	35	56	70	77
Good	30	48	65	73
Woods and grass combination (orchard or tree farm)				
Poor	57	73	82	86
Fair	43	65	76	82
Good	32	58	72	79
Woods				
Poor	45	66	77	83
Fair	36	60	73	79
Good	30	55	70	77
Farmsteads	59	74	82	86

The curve number method may also be used in determining the time distribution of the runoff. In this manual, the CN method is used in conjunction with the synthetic dimensionless and triangular unit hydrograph methods to determine the storm hydrograph. The procedure used in this operation is outlined below.

1. Determine the basin curve number.
2. Given the rainfall depth and storm duration, determine the time distribution of the rainfall. This distribution can be the SCS Type II or Huff Distributions discussed in Chapter 2.

3. Divide the storm duration into convenient intervals and determine the rainfall depth at the start and end of each interval.
4. Determine the ultimate abstraction S by using Equation 3.3.2.
5. Using Equation 3.3.1 or Figure 3.3.2, determine the runoff for the beginning and end of the time intervals.
6. The difference between successive runoff values is the incremental runoff. The incremental runoff may then be used as the excess rainfall needed as input in hydrograph calculations.

The curve number method was developed for agricultural watersheds. When applying the method to urban watersheds, discrepancies are observed for low AMC's when compared to other methods. (Burke and Gray, 1979).

These discrepancies arise because runoff begins too slowly in urbanized basins if a composite CN is used. To overcome this discrepancy, *the runoff should be calculated separately for the pervious and impervious areas and the resultant hydrographs added together.*

The SCS has developed two computer programs and the associated documentation to apply the CN method. The first model is the TR-20 program (SCS, 1982). This program was developed to generate runoff and simulate the movement of water through channels or reservoirs. TR-20 has been widely used for planning and design of water resource projects. It can be used to investigate the effects of varying storm durations, rainfall distributions, unit hydrograph, and AMC values. The TR-20 program is discussed in detail in Chapter 8.

The program TR-55 (SCS, 1986) provides users with a simplified procedure to evaluate the impacts of urbanization. In 1986, a computer version of TR-55 was published by the SCS which updated the CN methodologies and incorporated the latest research in the field. However, the program does not allow for the routing of flows and uses only the SCS Types I, II and III rainfall distributions.

The U.S. Army Corps of Engineers (COE) has developed HEC-1, a program similar to TR-20 (COE, 1987). Like TR-20, many of the inputs can be varied and channel and reservoir routings can be performed. HEC-1 also allows for the variation of the loss rate which is useful in model calibration and verification. This program is also discussed in detail in Chapter 8.

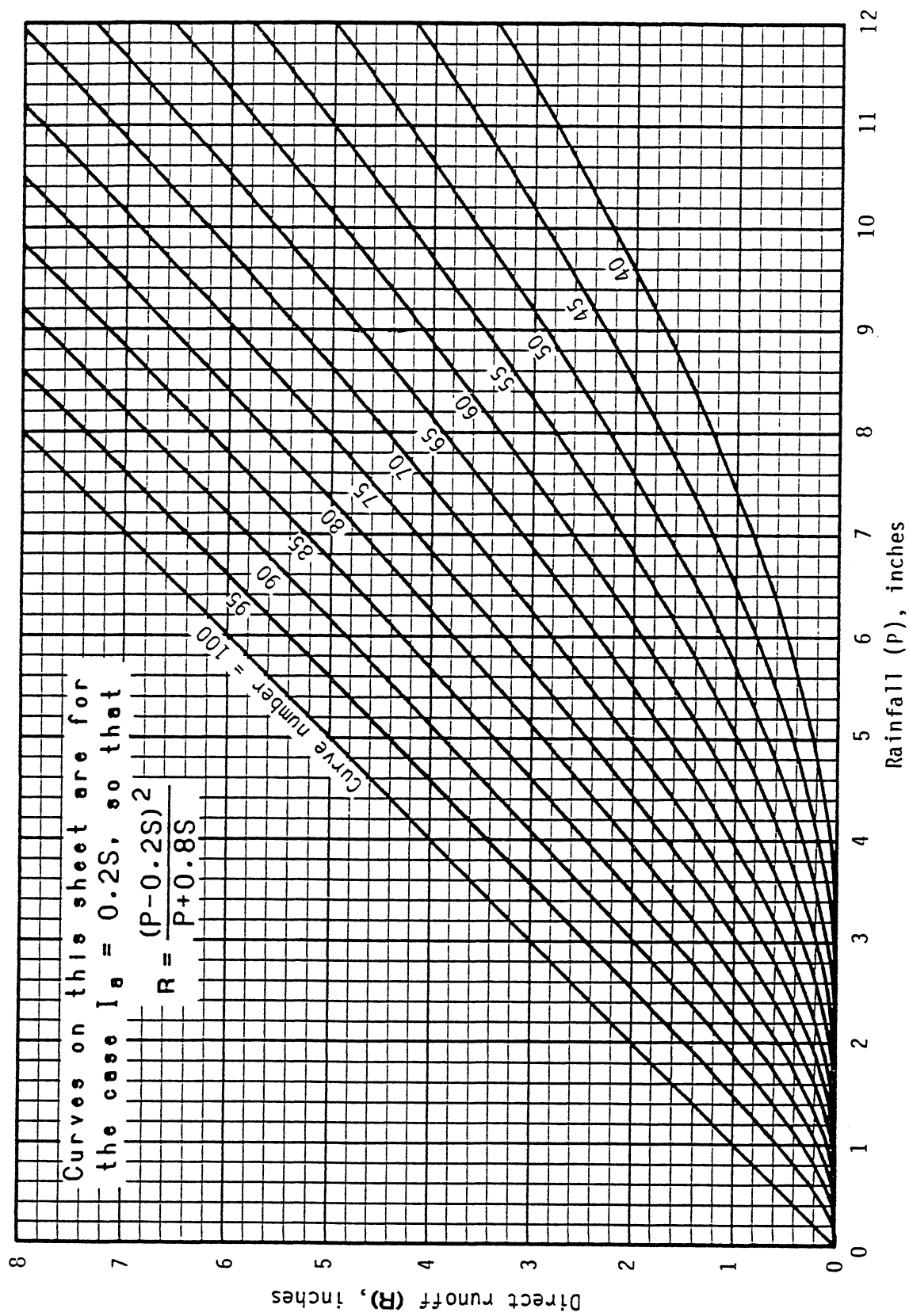


Figure 3.3.2
Graphical Solution of Equation 3.3.1 (SCS, 1986)

Example 3.3.1

This example illustrates the use of the SCS Curve Number Method

Determine the volume of runoff for a 50-year 2-hour storm in Indianapolis, Indiana with the following characteristics and antecedent moisture condition II.

Basin characteristics:

<u>Subbasin</u>	<u>Soil Type</u>	<u>Practice</u>	<u>Condition</u>	<u>Area (acres)</u>
1	Carmel	Cultivated Land w/o conservation treatment	Poor	55
2	Ockley	Meadow	Good	40
3	Hosmer	Residential; 1/8 acre	-----	25
4	Muskingum	Forest; good cover	Good	90

Step 1: Determine the soil type using Table 3.3.2 and the curve numbers using Table 3.3.3 to develop a composite curve number for the basin using Equation 3.3.3.

<u>Subbasin</u>	<u>Soil Classification</u>	<u>CN</u>
1	C	78
2	B	58
3	C	90
4	C	70

$$CN_{COMP} = \frac{55(78) + 40(58) + 25(90) + 90(70)}{210} = 72.19$$

Step 2: Determine the ultimate abstraction, S, using Equation 3.3.2.

$$S = \left(\frac{1000}{CN} \right) - 10 = \left(\frac{1000}{72.19} \right) - 10 = 3.85 \text{ inches}$$

Step 3: Determine the total rainfall for a 50-year return period and a 2-hour duration for Indianapolis, Indiana. From Equation 2.2.16 on page 2-30, the corresponding rainfall depth can be found.

$$i_2^{.50} = \frac{56.34 \log(4.11(50)^{0.386})}{(60(2) + 9.2)^{0.78}} = 1.61 \text{ inches/hr}$$

For a duration of 2-hours the depth is 3.22 inches.

Step 4: Determine the cumulative runoff $R(t)$ using Equation 3.3.1 or Figure 3.3.2.

$$R(t) = \frac{(P(t) - 0.2S)^2}{P(t) + 0.8S} = \frac{(3.22 - 0.2(3.85))^2}{3.22 + 0.8(3.85)} = 0.953 \text{ inches}$$

Step 5: Find the total runoff volume in cubic feet (ft^3).

$$0.953 \text{ inches} \cdot \frac{1 \text{ foot}}{12 \text{ inches}} \cdot 210 \text{ acres} \cdot \frac{43,560 \text{ ft}^2}{1 \text{ acre}} = 726,472 \text{ ft}^3$$

3.4 - HYDROGRAPHS

The physical processes of runoff and the factors which reduce or delay the amount of rainfall that becomes runoff, were discussed in Section 3.1.1. In the design of hydraulic structures, it is often necessary to know the time distribution of runoff as well as the peak magnitude. A hydrograph, which is simply a graph of flow versus time, enables both of these values to be determined.

A hydrograph may have four components: direct surface runoff, interflow, groundwater or base flow and channel precipitation. The initial rising portion of the hydrograph is termed as the rising limb, the portion in the vicinity of the peak is called the crest segment, and the falling limb is termed the recession curve (Figure 3.4.1). The shape of a hydrograph depends on the precipitation and the basin characteristics. (Linsley et al., 1975)

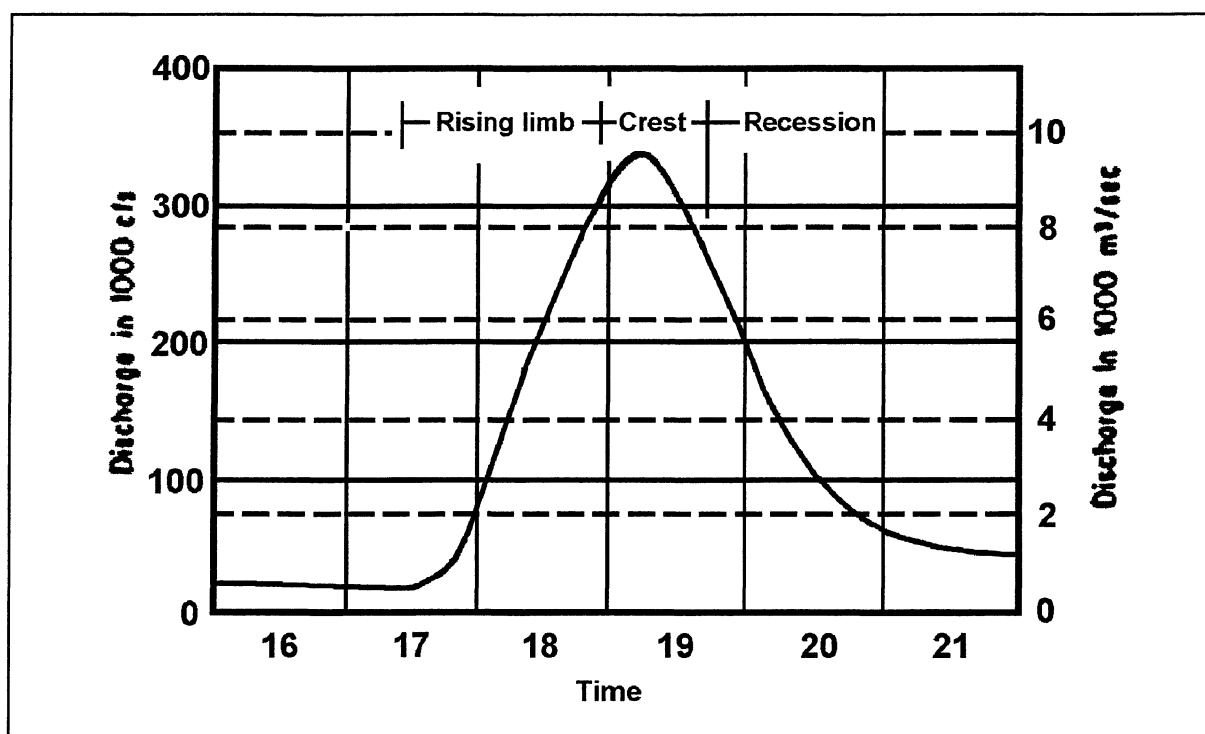


Figure 3.4.1
Definition of Hydrograph Terms (Linsley et al., 1975)

If rainfall remains at a constant intensity for a sufficiently long period of time, a state of equilibrium is reached. This point represents the time when the rate of rainfall equals the rate of runoff. In Figure 3.4.2, two curves are shown. The inflection point on curve A represents the time when the entire drainage basin is contributing to the flow. At this inflection point, the maximum storage capability of the basin is only partially filled. As the storm progresses, the state of equilibrium is reached and the curve levels off. In nature, the state of

equilibrium is seldom attained. This is due to different factors associated with the runoff process and the fact that rarely will the rain fall with a constant intensity for a long period of time. Curve B represents a single-peaked hydrograph resulting from a finite rain of duration t_r .

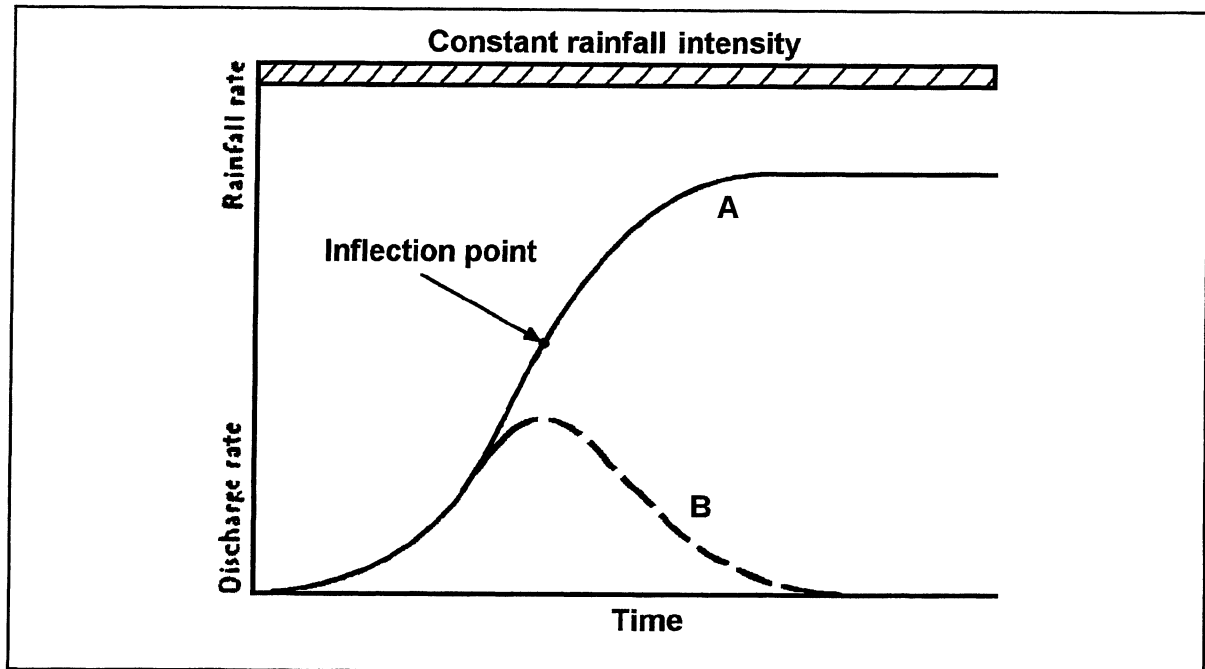


Figure 3.4.2
Equilibrium Discharge Hydrograph (Viessman et al., 1989)

There are four basic types of hydrographs (SCS; NEH-4):

- a. Natural Hydrograph: Obtained directly from the flow records of a gaged stream. These hydrographs can be used to calibrate computer models for both the peak flowrate and volume.
- b. Unit Hydrograph: A natural or synthetic hydrograph for one inch of direct runoff which is the amount of rain which becomes runoff after all losses and infiltration have been satisfied. The rainfall is assumed to occur uniformly over a watershed in a specified time.
- c. Dimensionless Unit Hydrograph: Represents many unit hydrographs by using the time to peak and the peak rates as basic units and plotting the hydrographs in ratios of these units. This is also called the Index Hydrograph.
- d. Synthetic Unit Hydrograph: A generalized unit hydrograph which contains adjustable parameters enabling it to be used for many watersheds.

The theory of the unit hydrograph and two synthetic hydrographs, the SCS Dimensionless Unit Hydrograph and the SCS Triangular Unit Hydrograph, are discussed in the following sections.

3.4.1 - The Unit Hydrograph

The techniques developed in this chapter provide a relationship between the rainfall input and the runoff so that hydrographs can be estimated for a particular basin with various temporal rainfall distributions. The unit hydrograph is defined as the hydrograph of direct runoff resulting from one inch of effective rainfall uniformly distributed in space and time over the watershed for a particular excess rainfall duration. The result is a hydrograph which is associated with a particular basin of constant physical conditions. *Direct Runoff* is the portion of the rainfall which finds its way to a channel and eventually reaches the point under consideration not long after the storm. This would include overland flow, interflow and groundwater. *Excess Rainfall* is the rainfall that actually contributes to direct runoff. Consequently, losses such as infiltration, evaporation and temporary or permanent storage are subtracted from the total rainfall.

The important concept behind the unit hydrograph theory is that the volume of excess rainfall equals the volume of runoff. The volume of runoff is the area under the hydrograph while the volume of excess rainfall is the depth times the area on which it falls. The depth in the unit hydrograph is one inch. In other words, the area under a hydrograph is equivalent to one inch of excess rainfall.

Once a unit hydrograph is computed, a rainfall depth corresponding to a storm of the same duration as the unit hydrograph may be developed. For example, a ten-minute unit hydrograph would be calculated on the depth of rainfall occurring in ten minutes. That depth is then multiplied by the unit hydrograph coordinates to develop the storm hydrograph associated with that rainfall depth. Two or more hydrographs, representing successive rainfall increments for a given time interval during the same storm, may be superimposed to determine the resulting storm hydrograph.

The unit hydrograph concept assumes that the volume of runoff is constant regardless of the temporal distribution. In nature, for the same rainfall depth, an increase in the precipitation duration lengthens the base and lowers the peak of the hydrograph. Normally this phenomenon changes the unit hydrograph only slightly. These changes are assumed to be negligible.

The unit hydrograph may be computed in several ways. The historical record of discharges may be analyzed to determine the unit hydrograph if a long streamflow record is available. Another method is to use synthetic unit hydrographs which have been developed by analyzing hydrographs from many basins.

3.4.2 - SCS Synthetic Unit Hydrographs

Two synthetic unit hydrographs are discussed in this section: the dimensionless unit hydrograph and the triangular unit hydrograph. Both of these incorporate empirical equations which relate the peak flow rate and the time to peak to basin parameters. These unit hydrographs were developed by the Soil Conservation Service and are based on the Curve Number Method in the calculation of excess precipitation.

The dimensionless unit hydrograph shown in Figure 3.4.3 was developed by analyzing data from a large number of basins with varying geographic locations. Dimensionless ratios of q/q_p on the ordinate and t/t_p on the abscissa where q_p and t_p are the peak flow and time to peak, respectively, are used to specify the hydrograph. Table 3.4.1 lists the coordinates of the hydrograph as well as the cumulative mass curve which is also shown in Figure 3.4.3. 37.5% of the total runoff volume accumulates in the rising portion of the hydrograph.

The unit hydrograph may be represented as a triangle. Both the triangular unit hydrograph and curvilinear unit hydrograph are shown in Figure 3.4.4. If we say that t_p (the time to peak) is "one time unit" we can determine t_b , the base time of the triangular hydrograph, to be $2.67 t_p$. It follows that the recession duration t_r is equal to $1.67 t_p$.

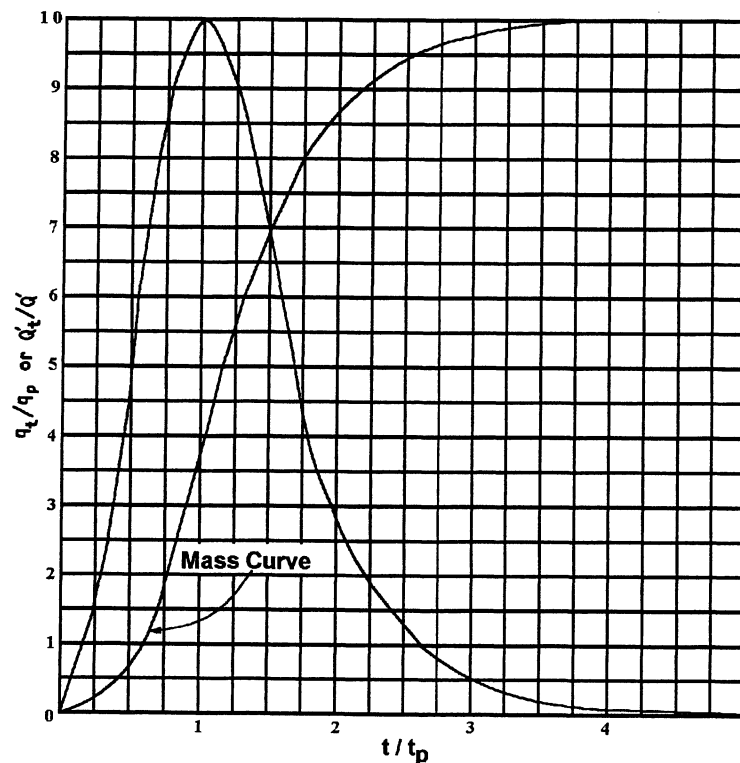


Figure 3.4.3
Dimensionless Unit Hydrograph and Mass Curve (SCS; NEH-4)

Table 3.4.1
Ratios for Dimensionless Unit Hydrograph
and Mass Curve (SCS; NEH-4)

<u>Time Ratios</u>	<u>Discharge Ratios</u>	<u>Mass Curve Ratios</u>
0.0	.000	.000
0.1	.030	.001
0.2	.100	.006
0.3	.190	.012
0.4	.310	.035
0.5	.470	.065
0.6	.660	.107
0.7	.820	.163
0.8	.930	.228
0.9	.990	.300
1.0	1.000	.375
1.1	.990	.450
1.2	.930	.522
1.3	.860	.589
1.4	.780	.650
1.5	.680	.700
1.6	.560	.751
1.7	.460	.790
1.8	.390	.822
1.9	.330	.849
2.0	.280	.871
2.2	.207	.908
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.6	.029	.989
3.8	.021	.993
4.0	.011	.997
4.5	.005	.999
5.0	.000	1.000

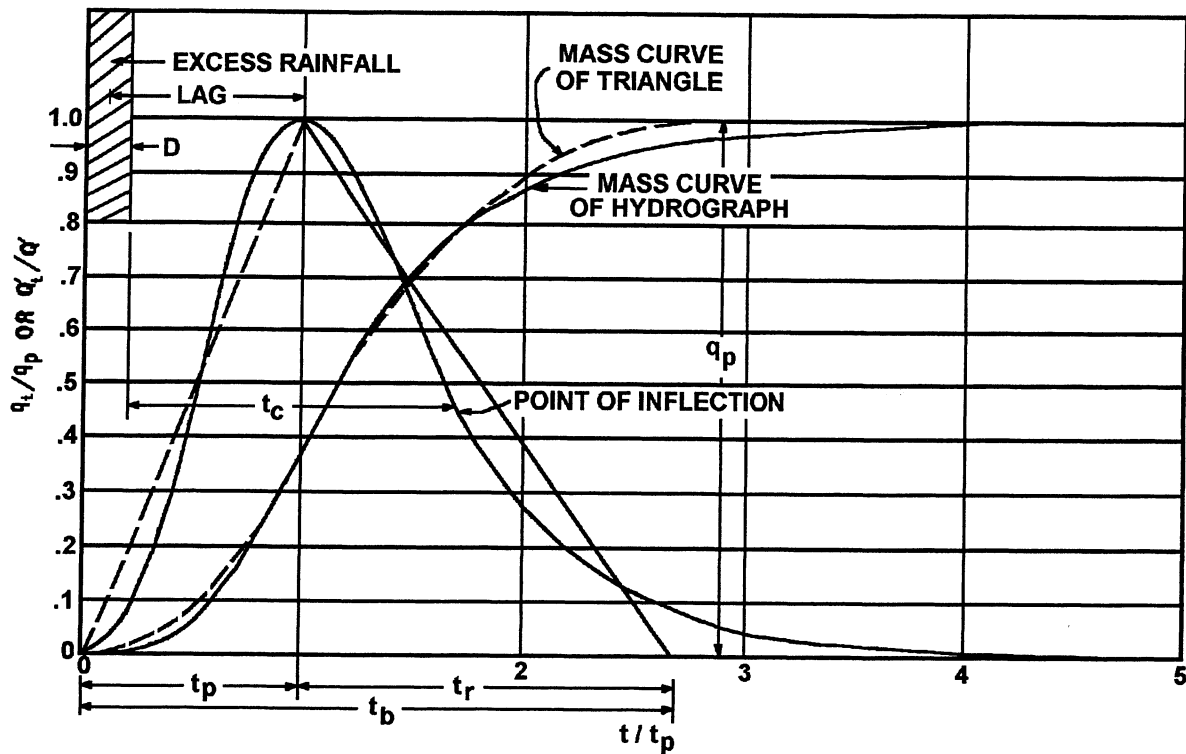


Figure 3.4.4
Dimensionless Curvilinear Unit Hydrograph and
Equivalent Triangular Hydrograph (SCS; NEH-4)

Other relationships associated with the unit hydrograph are as follows. The total volume of the triangular unit hydrograph Q' in Figure 3.4.4 may be expressed as:

$$V_t = Q' = \frac{t_p q_p}{2} + \frac{t_r q_p}{2} \quad (3.4.1)$$

where Q' is in inches, t is in hours, and q_p has units of inches/hour. Equation 3.4.1 can be manipulated to give Equation 3.4.2 where q_p is in cfs, A_m is in mi^2 and Q' is in inches.

$$q_p = \frac{484A_m Q'}{t_p} \quad (3.4.2)$$

It has been observed by the SCS that the watershed lag (L') is related to the t_c of the basin as $L' = 0.6 t_c$. Consequently, the time to peak (t_p) may be expressed in terms of the duration of unit excess rainfall (ΔD) and L' as shown in Equation 3.4.3.

$$t_p = \frac{\Delta D}{2} + L' = \frac{\Delta D}{2} + 0.6 t_c \quad (3.4.3)$$

$$\therefore q_p = \frac{484A_m Q'}{t_p} = \frac{484A_m Q'}{\frac{\Delta D}{2} + 0.6 t_c} \quad (3.4.4)$$

The dimensionless unit hydrograph, Figure 3.4.3, has a point of inflection at approximately $1.7t_p$ and can be expressed in terms of t_c and ΔD as; $t_c + \Delta D = 1.7t_p$. Substituting t_p found in Equation 3.4.3 into $t_c + \Delta D = 1.7t_p$ and simplifying gives $\Delta D = 0.133t_c$. A small variation in ΔD is permissible, but it should not exceed $0.17t_p$ (Viessman et al., 1989).

3.4.3 - Comparison of Unit Hydrographs

The preceding relationships allow the development of the dimensionless unit hydrograph as follows:

Step 1: Find the time of concentration, t_c . This value may be obtained using Figure 3.4.5, in which the ordinate is entered with the travel path slope, then the diagonal line which represents either a paved or unpaved surface is intercepted, and a velocity is found by reading the abscissa. The travel time, t_t , is then found by using Equation 3.4.5. The time of concentration, t_c , is the sum of t_t values for various consecutive flow segments.

$$t_t = \frac{L}{3600 v} \quad t_c = t_{t1} + t_{t2} + \dots t_{tm} \quad (3.4.5)$$

Step 2: Determine the values of t_p , and q_p from Equations 3.4.3 and 3.4.4, where Q is equal to one for a unit hydrograph and ΔD is the rainfall duration.

$$q_p = \frac{484A_m Q'}{t_p} \quad t_p = \frac{\Delta D}{2} + L' = \frac{\Delta D}{2} + 0.6t_c$$

Step 3: Determine the coordinates of the unit hydrograph. By using Table 3.4.1, the values of discharge and time may be found by multiplying the ratios by q_p and t_p . These may be plotted to develop the unit hydrograph.

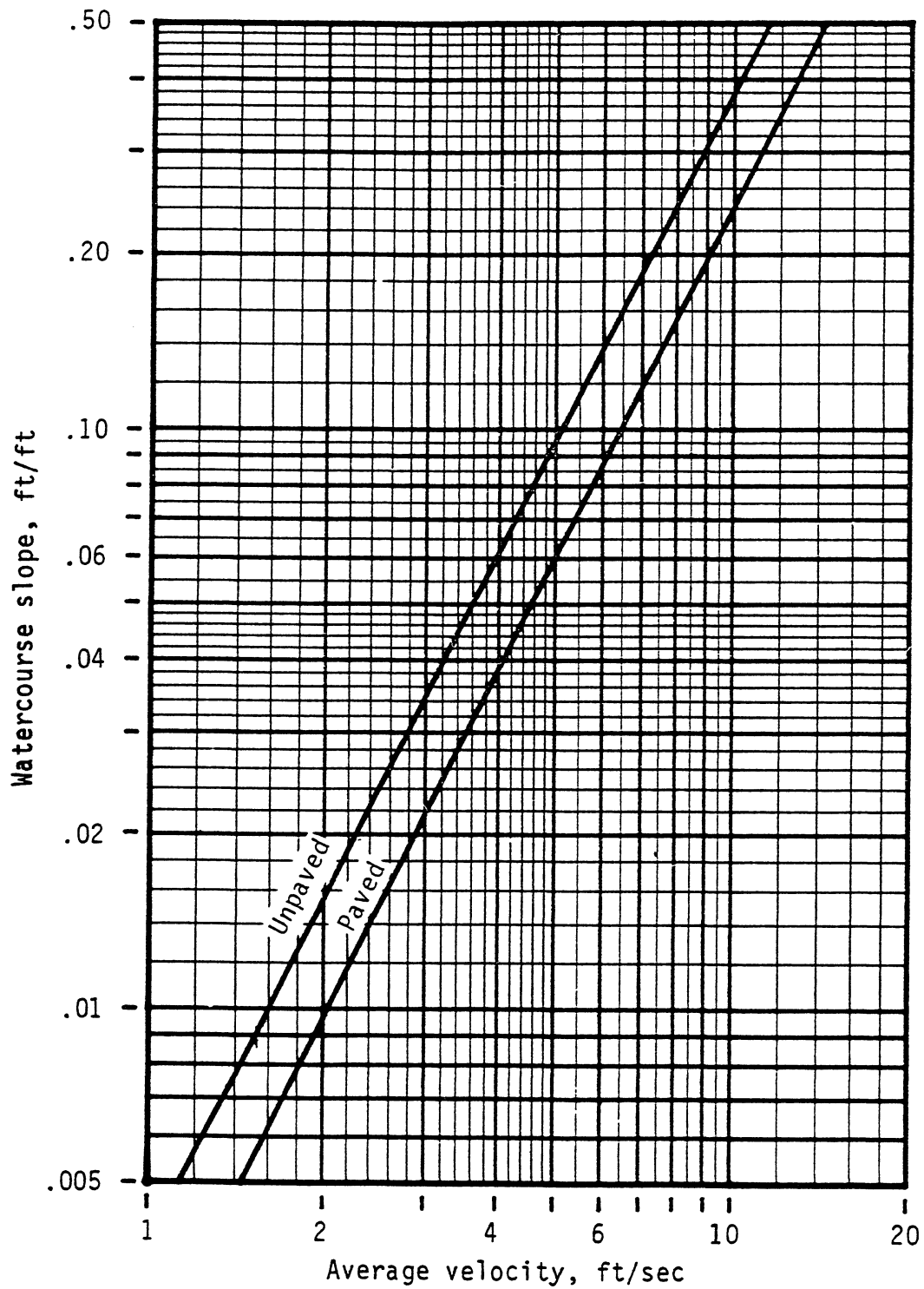


Figure 3.4.5
Average Velocities for Estimating Travel Time (SCS, 1986)

Step 4: Check that the volume of the unit hydrograph is equal to 1 inch of excess rainfall falling on the basin. This is accomplished by summing the coordinates of q_t and multiplying by ΔD . This may be expressed as:

$$V_{UH} = \Delta D \sum_{t=1}^{\left(\frac{5 t_p}{\Delta D}\right)} q_t \quad (3.4.6)$$

where V_{UH} has units of cfs-hr.

The value, V_{UH} , obtained in Equation 3.4.5 should be compared to the volume, V , obtained by Equation 3.4.6:

$$V = A_m * 645.33 \quad (3.4.7)$$

Where V has units of cfs-hour and A_m has units of mi^2 .

The results from Equations 3.4.5 and 3.4.6 should be very close. If there is a difference, the ordinates of the unit hydrograph should be adjusted.

The triangular unit hydrograph may be determined by using Step 2 outlined above and the relationship $t_b = 2.67 t_p$.

3.4.4 - Storm Hydrographs

Once a unit hydrograph has been determined, the hydrograph for a given temporal distribution of a storm and basin characteristics can be determined as follows.

Before a hydrograph can be developed, the basin area (sq. mi.), basin curve number (see Section 3.3), antecedent moisture condition (see Section 3.3), storm duration, depth, and temporal distribution must be known. The following steps outline the procedure used in the application of dimensionless unit hydrograph to determine the storm hydrograph from a unit hydrograph.

Step 1: Tabulate the cumulative rainfall at ΔD intervals. Tabulate the unit hydrograph at ΔD time intervals.

Step 2: Using Equation 3.3.1 calculate the cumulative runoff using the curve number (see Section 3.3.3).

Step 3: Calculate the incremental runoff for each time interval.

Step 4: The storm hydrograph may be found using the results of Step 1, Step 3 and Equation 3.4.8.

$$Q_t = \sum_{i=\mu}^v (u_{t-i+1}) (q_i) \quad t = 1, \dots, NP + NU - 1 \quad (3.4.8)$$

This is valid for $NP < NU$.

where Q_t is the runoff at time t (cfs), u is the ordinate of the unit hydrograph [cfs/inch], q is the excess rainfall [inches], NP is the number of elements in the rainfall hyetograph, NU is the number of coordinates in the unit hydrograph, μ is $\max [1, t+1-NU]$, and v is $\min [t, NP]$.

These equations may be solved with a computer program such as TR-20 or HEC-1 or a small programmable calculator (Croley, 1977). They may also be solved manually as outlined in Example 3.4.1. The time interval for the hydrograph is assumed to be ΔD and it is assumed that $NP < NU$.

Step 5: Compute the volume of the new hydrograph by summing the ordinates and multiplying by the time interval. Compare this to the actual volume (area x accumulated runoff).

Example 3.4.1

A basin in Fort Wayne, IN has an area of 0.47 mi^2 and a weighted curve number of 84. Determine the 2 hour - 50 year storm hydrograph by using a unit hydrograph. Assume that the water travels 9000 ft. on unpaved surface with a 1.6% slope.

Step 1: Find the time of concentration, t_c , using Figure 3.4.5. Enter the ordinate with a 1.6% slope and go across until the unpaved line is intersected and a velocity of 2.0 ft/sec is found. The time of concentration is equal to

$$t_c = \frac{L}{v} = \frac{9000 \text{ ft}}{2 \text{ ft/sec}} = 4500 \text{ seconds} = 75.0 \text{ minutes}$$

Step 2: Determine the values of ΔD , t_p and q_p . From page 3-31:
 $\Delta D = 0.133t_c = 0.133(75.0) = 9.98 \text{ minutes} \approx 10 \text{ minutes}$
 $t_p = (\Delta D/2) + 0.6t_c = (10/2) + 0.6(75.0) = 0.83 \text{ hours}$

$$q_p = \frac{484A_m Q'}{t_p} = \frac{484(0.47 \text{ mi}^2)(1)}{0.83 \text{ hours}} = 274 \text{ cfs}$$

Step 3: Determine the coordinates of the dimensionless unit hydrograph using Table 3.4.1. For this example, ΔD is 5 minutes.

<u>Time Ratio</u>	<u>Time</u>	<u>Discharge Ratio</u>	<u>Discharge</u>
t / t_p	t (min.)	(q / q_p)	q (cfs)
0.0	0	0.000	0
0.1	5	0.030	8.22
0.2	10	0.100	27.40
0.3	15	0.190	52.06
0.4	20	0.310	84.94
0.5	25	0.470	128.78
0.6	30	0.660	180.84
0.7	35	0.820	224.68
0.8	40	0.930	254.82
0.9	45	0.990	271.26
1.0	50	1.000	274.00
1.1	55	0.990	271.26
1.2	60	0.930	254.82
1.3	65	0.860	235.64
1.4	70	0.780	213.72
1.5	75	0.680	186.32
1.6	80	0.560	153.44
1.7	85	0.460	126.04
1.8	90	0.390	106.86
1.9	95	0.330	90.42
2.0	100	0.280	76.72
2.2	110	0.207	56.72
2.4	120	0.147	40.28
2.6	130	0.107	29.32
2.8	140	0.077	21.10
3.0	150	0.055	15.07
3.2	160	0.040	10.96
3.4	170	0.029	7.95
3.6	180	0.021	5.75
3.8	190	0.015	4.11
4.0	200	0.011	3.01
4.5	225	0.005	1.37
5.0	250	0.000	0.00

The values of t and q are plotted to get the unit hydrograph shown in Figure 3.4.6.

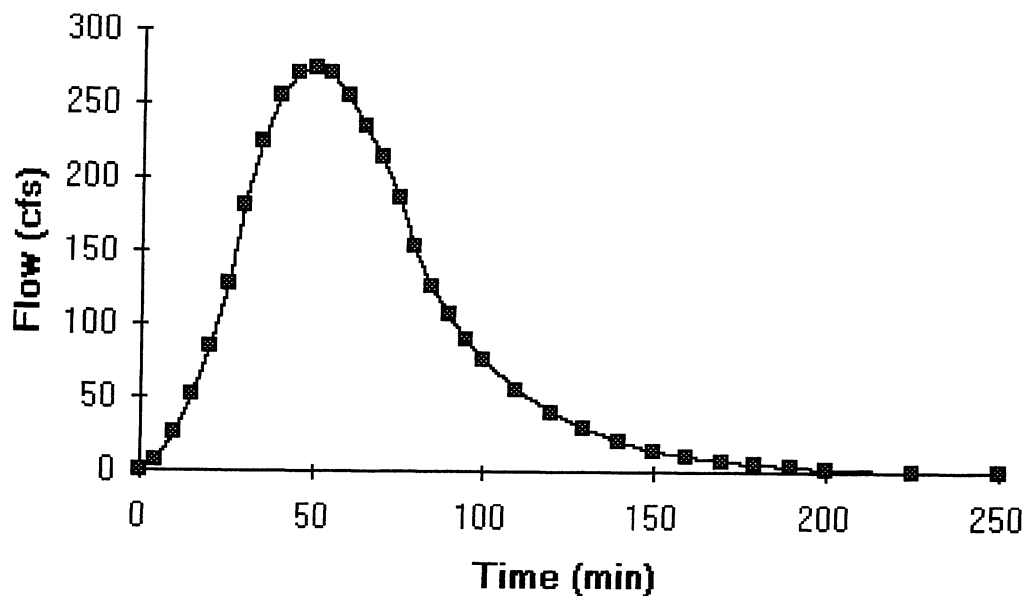


Figure 3.4.6
Dimensionless Unit Hydrograph for Example 3.4.1

Step 4: Check to see that the volume of the unit hydrograph is equal to 1 inch. Sum the unit hydrograph ordinates.

$$(0 + 8.22 + 27.40 + \dots + 1.50 + 1.37 + 1.00) \times 5 = 18,377 \text{ cfs-min. } (*)$$

*Note: Discharges for Time Ratio greater than 4.5 are considered zero.

$$18,337 \frac{\text{ft}^3 \cdot \text{min}}{\text{sec}} \cdot \frac{60 \text{ sec}}{\text{min}} \cdot \frac{\text{acre}}{43,560 \text{ ft}^2} = 25.26 \text{ acre-feet}$$

$$1 \text{ inch} \cdot \frac{1 \text{ ft}}{12 \text{ inches}} \cdot 300 \text{ acres} = 25.00 \text{ acre-feet}$$

$$\frac{25.26 - 25.00}{25.26} \cdot 100\% = 1.03\% \text{ difference}$$

Once the unit hydrograph has been established, the storm hydrograph is computed by using the temporal distribution of the rainfall.

Step 5: Find the rainfall depth. Using Equation 2.2.13 with the coefficients in Table 2.2.2, the 2-hour, 50-year rainfall intensity for South Bend is 1.36 in/hr. The 2-hour, 50-year rainfall depth is 2.72 in.

From Table 2.2.2, with a recurrence interval of 50-years and a 2-hour duration, the coefficients for South Bend are:

$$c = 1.2799 \quad \alpha = 0.1872 \quad d = 0.258 \quad \beta = 0.8252$$

The corresponding rainfall intensity is:

$$i = \frac{c T_r^\alpha}{(t + d)^\beta} = \frac{1.2799 (50)^{0.1872}}{(2 + 0.258)^{0.8252}} = 1.36 \text{ inches/hour}$$

For the 2-hour duration, the rainfall depth is $2 \times 1.36 = 2.72$ inches.

Tabulate the accumulated runoff at 10-minute intervals for the total rainfall depth of 2.72 inches using Huff's first quartile median distribution curve from Table 2.1.5.

Time (min)	Cum % of Total Storm Time	Cum % of Total Storm Rainfall	Cum Rainfall Depth (inch)	Cum Runoff R(t) (inch)*	Increm Runoff R(t) (inch)
0	0	0	0 00	0 00	
10	8 33	16 66	0 45	0 00	0 00
20	16 67	33 34	0 91	0 11	0 11
30	25 00	45 84	1 25	0 27	0 16
40	33 33	54 74	1 49	0 41	0 14
50	41 67	61 97	1 69	0 53	0 12
60	50 00	67 35	1 83	0 63	0 10
70	58 33	73 72	2 01	0 75	0 12
80	66 67	78 89	2 15	0 85	0 10
90	75 00	83 75	2 28	0 95	0 10
100	83 33	88 74	2 41	1 05	0 10
110	91 67	94 08	2 56	1 16	0 11
120	100 00	100 00	2 72	1 29	0 13

$$* R(t) = \left(\frac{(P(t) - 0.2S)^2}{P(t) + 0.8 S} \right) \quad \text{Where } S = \left(\frac{1000}{84} \right) - 10 = 1.90 \text{ inch, } R(t) = 0 \text{ if } P(t) < 0.2S$$

Step 6: Compute and construct the storm hydrograph. Multiplying the unit hydrograph in Figure 3.4.6 by the incremental runoff and summing values across as shown in Figure 3.4.7, the storm hydrograph presented in Figure 3.4.8 is developed. A blank form for the calculations is given in Figure 3.4.9.

Time (min)	Unit Hydrograph (cfs)	Excess Precipitation (inches)														Storm Hydrograph (cfs)
		0.00	0.11	0.16	0.14	0.12	0.10	0.12	0.10	0.10	0.10	0.10	0.11	0.13		
0	0.00	0.00													0.00	
10	27.40	0.00	0.00												0.00	
20	84.94	0.00	3.01	0.00											3.01	
30	180.84	0.00	9.34	4.38	0.00										13.73	
40	254.82	0.00	19.89	13.59	3.84	0.00									37.32	
50	274.00	0.00	28.03	28.93	11.89	3.29	0.00								72.14	
60	254.82	0.00	30.14	40.77	25.32	10.19	2.74	0.00							109.16	
70	213.72	0.00	28.03	43.84	35.67	21.70	8.49	3.29	0.00						141.03	
80	153.44	0.00	23.51	40.77	38.36	30.58	18.08	10.19	2.74	0.00					164.24	
90	106.86	0.00	16.88	34.20	35.67	32.88	25.48	21.70	8.49	2.74	0.00				178.05	
100	76.72	0.00	11.75	24.55	29.92	30.58	27.40	30.58	25.48	2.74	0.00				184.10	
110	56.72	0.00	8.44	17.10	21.48	25.65	25.48	32.88	25.48	8.49	3.01	0.00			186.10	
120	40.28	0.00	6.24	12.28	14.96	18.41	21.37	30.58	27.40	18.08	9.34	3.56			181.71	
130	29.32	0.00	4.43	9.08	10.74	12.82	15.34	25.65	25.48	27.40	25.48	19.89	11.04		187.36	
140	21.10	0.00	3.23	6.44	7.94	9.21	10.69	18.41	21.37	25.48	27.40	28.03	23.51		181.71	
150	15.07	0.00	2.32	4.69	5.64	6.81	7.67	12.82	15.34	21.37	25.48	30.14	33.13		165.42	
160	10.96	0.00	1.66	3.38	4.10	4.83	5.67	9.21	10.69	15.34	21.37	28.03	35.62		139.90	
170	7.95	0.00	1.21	2.41	2.95	3.52	4.03	6.81	7.67	10.69	15.34	23.51	33.13		111.26	
180	5.75	0.00	0.87	1.75	2.11	2.53	2.93	4.83	5.67	7.67	10.69	16.88	27.78		83.73	
190	4.11	0.00	0.63	1.27	1.53	1.81	2.11	3.52	4.03	5.67	7.67	11.75	19.95		59.95	
200	3.01	0.00	0.45	0.92	1.11	1.32	1.51	2.53	2.93	4.03	5.67	8.44	13.89		42.80	
210	2.50	0.00	0.33	0.66	0.81	0.95	1.10	1.81	2.11	2.93	4.03	6.24	9.97		30.93	
220	2.00	0.00	0.28	0.48	0.58	0.69	0.80	1.32	1.51	2.11	2.93	4.43	7.37		22.49	
230	1.50	0.00	0.22	0.40	0.42	0.49	0.58	0.95	1.10	1.51	2.11	3.23	5.24		16.24	
240	1.00	0.00	0.17	0.32	0.35	0.36	0.41	0.69	0.80	1.10	1.51	2.32	3.81		11.83	
250	0.00	0.00	0.11	0.24	0.28	0.30	0.30	0.49	0.58	0.80	1.10	1.66	2.74		8.59	
260			0.00	0.16	0.21	0.24	0.25	0.36	0.41	0.58	0.80	1.21	1.96		6.17	
270				0.00	0.14	0.18	0.20	0.30	0.41	0.58	0.80	0.87	1.42		4.41	
280					0.00	0.12	0.15	0.24	0.25	0.30	0.41	0.63	1.03		3.14	
290						0.00	0.10	0.18	0.20	0.25	0.30	0.45	0.75		2.23	
300							0.00	0.12	0.15	0.20	0.25	0.33	0.53		1.59	
310								0.00	0.10	0.15	0.20	0.28	0.39		1.12	
320									0.00	0.10	0.15	0.22	0.33		0.80	
330										0.00	0.10	0.17	0.26		0.53	
340											0.00	0.11	0.20		0.31	
350												0.00	0.13		0.13	
360													0.00		0.00	

Figure 3.4.7
Computation of the Storm Hydrograph for Example 3.4.1

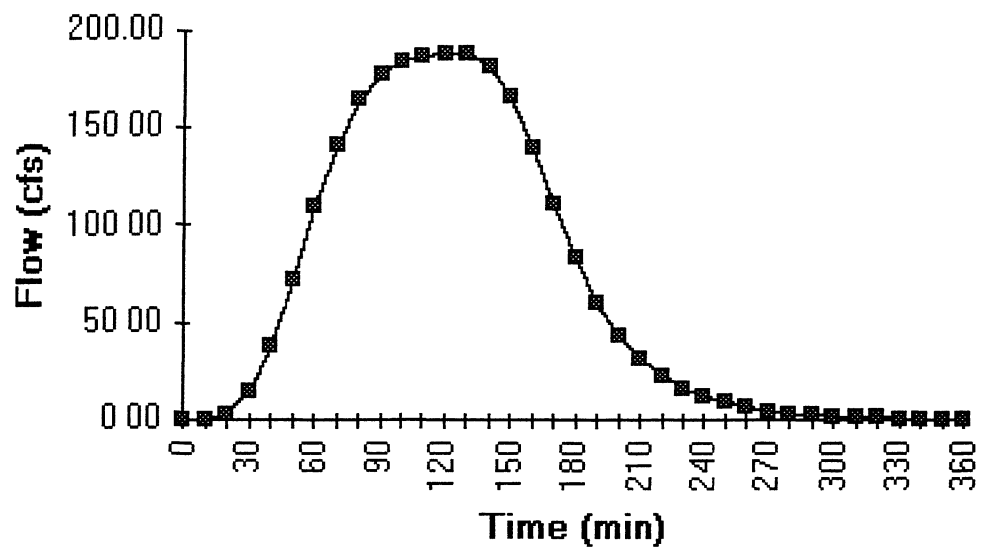


Figure 3.4.8
Storm Hydrograph for Example 3.4.1

3.5 - WATER RESOURCES COUNCIL METHOD

The use of the Water Resources Council Method (WRC) for flood flow frequency analysis has been mandated for projects involving Federal funds. The procedure for flood frequency analysis by using the WRC method and annual maximum flows x_i , $i=1, 2, \dots, n$, is as follows (U.S. Water Resources Council, 1981).

Step 1: Convert x_i into logarithmic values, y_i .

$$y_i = \log (x_i)$$

Step 2: Compute the sample statistics

$$MEAN = \bar{y} = \frac{1}{n} \sum_{i=1}^n y_i$$

$$STANDARD \ DEVIATION = s_y = \left(\frac{1}{n-1} \sum_{i=1}^n (y_i - \bar{y})^2 \right)^{\frac{1}{2}}$$

$$SKEWNESS \ COEFFICIENT = C_s = \frac{n \sum_{i=1}^n (y_i - \bar{y})^3}{(n-1)(n-2) s_y^3}$$

Step 3: Test for outliers. These are flowrates which are too high or low to be reconciled with the remaining data points. Inclusion of these data points will significantly affect the sample statistics, and thus the final results. The logarithms of high and low outlier thresholds are found by:

$$y_H = \bar{y} + K_n \cdot s_y \qquad y_L = \bar{y} - K_n \cdot s_y$$

where the values of K_n are shown in Table 3.5.1 and are used in *one-sided tests* that detect outliers at the 10-percent level of significance in normally distributed data. According to the Water Resources Council (1981), if information is available that indicates a high outlier is the maximum over an extended period of time, the outlier is treated as historic flood data and excluded from analysis. If useful historic information is not available to compare to high outliers, then the outliers should be retained as part of the systematic record. Flood peaks considered low outliers are deleted from the procedure is repeated.

Step 4: Consult Figure 3.5.1 to determine the map skew, C_m . Most of Indiana has a map skew of -0.4.

Table 3.5.1
Values of K_n for Various Sample Sizes

Sample Size, n	K_n	Sample Size, n	K_n
10	2.036	35	2.628
11	2.088	36	2.639
12	2.134	37	2.650
13	2.175	38	2.661
14	2.213	39	2.671
15	2.247	40	2.682
16	2.279	41	2.692
17	2.309	42	2.700
18	2.335	43	2.710
19	2.361	44	2.719
20	2.385	45	2.727
21	2.408	50	2.768
22	2.429	55	2.804
23	2.448	60	2.837
24	2.467	65	2.866
25	2.486	70	2.893
26	2.502	75	2.917
27	2.519	80	2.940
28	2.534	85	2.961
29	2.549	90	2.981
30	2.563	95	3.000
31	2.577	100	3.017
32	2.591	110	3.049
33	2.604	120	3.078
34	2.616	130	3.104
		140	3.129

Step 5: Determine the values of G and H, where:

$$G = -0.33 + 0.08 |C_s| \quad \text{if} \quad |C_s| \leq 0.9$$

$$G = -0.52 + 0.30 |C_s| \quad \text{if} \quad |C_s| > 0.9$$

$$H = 0.94 - 0.26 |C_s| \quad \text{if} \quad |C_s| \leq 1.50$$

$$H = 0.55 \quad \text{if} \quad |C_s| > 1.50$$

Step 6: Determine the Variance of the station skew, $V(C_s)$.

$$V(C_s) = 10^{G - H \log(n/10)}$$

Step 7: Determine the weight, W , to place on the station skew.

$$W = \frac{V(C_m)}{V(C_s) + V(C_m)}$$

$V(C_m)$, the coefficient of map skew, has been estimated to be equal to 0.303.

Step 8: Determine the weighted skew of the station.

$$C_w = W \cdot C_s + (1 - W) \cdot 0.303$$

Step 9: Determine the frequency factors, K_p , corresponding to the various return periods using Table 3.12.

Step 10: Determine flow rates for the various return periods using the following relationships:

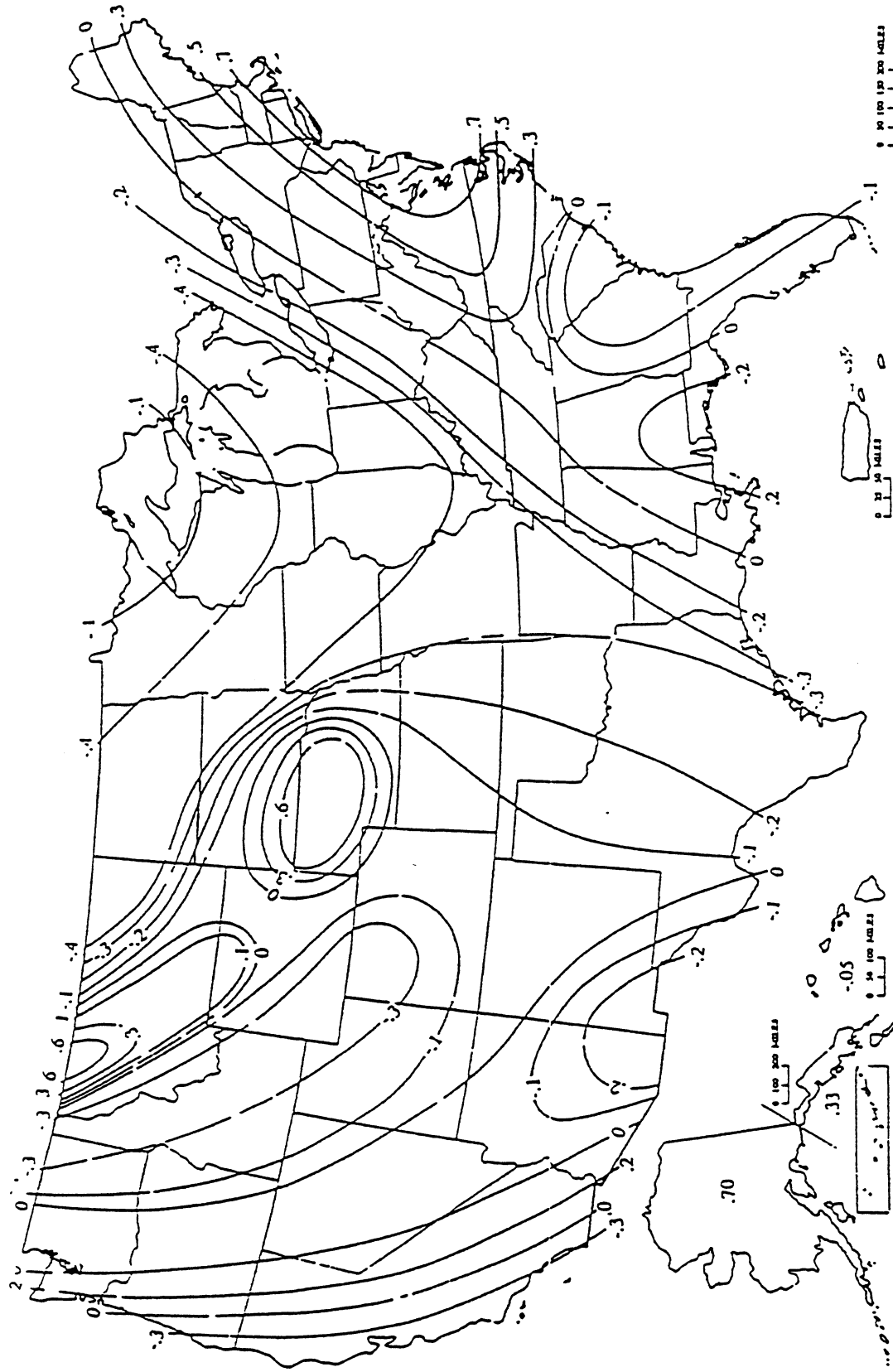
$$y_{T_r} = y + K_T s_y$$

$$Q_{T_r} = 10^{y_{T_r}}$$

A computer program is available to perform the flood frequency analysis computations and plot the desired results with confidence intervals. The HEC-FFA program, formerly known as HECWRC, computes the flood frequencies in accordance with "Guidelines for Determining Flood Flow Frequencies," Bulletin 17B of the U.S. Water Resources Council (WRC), March 1982. The program and user's manual is available from the U.S. Army Corps of Engineers, (916) 756-1104 (HEC-FFA, 1992).

Table 3 5.2
K_T Values For Water Resource Council Method
(Log Pearson Type III Distribution)
Return Period in Years

Skew Coefficient, C _s	2	5	10	25	50	100
-3.0	0.396	0.636	0.666	0.666	0.666	0.667
-2.9	0.390	0.651	0.681	0.683	0.689	0.690
-2.8	0.384	0.666	0.702	0.712	0.714	0.714
-2.7	0.376	0.681	0.724	0.738	0.740	0.740
-2.6	0.368	0.696	0.747	0.764	0.768	0.769
-2.5	0.360	0.711	0.771	0.793	0.798	0.799
-2.4	0.351	0.725	0.795	0.823	0.830	0.832
-2.3	0.341	0.739	0.819	0.855	0.864	0.867
-2.2	0.330	0.752	0.844	0.888	0.900	0.905
-2.1	0.319	0.765	0.869	0.923	0.939	0.946
-2.0	0.307	0.777	0.895	0.959	0.980	0.990
-1.9	0.294	0.788	0.920	0.996	1.023	1.037
-1.8	0.282	0.799	0.945	1.035	1.069	1.087
-1.7	0.268	0.808	0.970	1.075	1.116	1.140
-1.6	0.254	0.817	0.994	1.116	1.166	1.197
-1.5	0.240	0.825	1.018	1.157	1.217	1.256
-1.4	0.225	0.832	1.041	1.198	1.270	1.318
-1.3	0.210	0.838	1.064	1.240	1.324	1.383
-1.2	0.195	0.844	1.086	1.282	1.379	1.449
-1.1	0.180	0.848	1.107	1.324	1.435	1.518
-1.0	0.164	0.852	1.128	1.366	1.492	1.588
-0.9	0.148	0.854	1.147	1.407	1.549	1.660
-0.8	0.132	0.856	1.166	1.448	1.606	1.733
-0.7	0.116	0.857	1.183	1.488	1.663	1.806
-0.6	0.099	0.857	1.200	1.528	1.720	1.880
-0.5	0.083	0.856	1.216	1.567	1.777	1.955
-0.4	0.066	0.855	1.231	1.606	1.834	2.029
-0.3	0.050	0.853	1.245	1.643	1.890	2.104
-0.2	0.033	0.850	1.258	1.680	1.945	2.178
-0.1	0.017	0.846	1.270	1.716	2.000	2.252
0.0	0.000	0.842	1.282	1.751	2.054	2.326
0.1	-0.017	0.836	1.292	1.785	2.107	2.400
0.2	-0.033	0.830	1.301	1.818	2.159	2.472
0.3	-0.050	0.824	1.309	1.849	2.211	2.544
0.4	-0.066	0.816	1.317	1.880	2.261	2.615
0.5	-0.083	0.808	1.323	1.910	2.311	2.686
0.6	-0.099	0.800	1.328	1.939	2.359	2.755
0.7	-0.116	0.790	1.333	1.967	2.407	2.824
0.8	-0.132	0.780	1.336	1.993	2.453	2.891
0.9	-0.148	0.769	1.339	2.018	2.498	2.957
1.0	-0.164	0.758	1.340	2.043	2.542	3.022
1.1	-0.180	0.745	1.341	2.066	2.585	3.087
1.2	-0.195	0.732	1.340	2.087	2.626	3.149
1.3	-0.210	0.719	1.339	2.108	2.666	3.211
1.4	-0.225	0.705	1.337	2.128	2.706	3.271
1.5	-0.240	0.690	1.333	2.146	2.743	3.330
1.6	-0.254	0.675	1.329	2.163	2.780	3.388
1.7	-0.268	0.660	1.324	2.179	2.815	3.444
1.8	-0.282	0.643	1.318	2.193	2.848	3.499
1.9	-0.294	0.627	1.310	2.207	2.881	3.553
2.0	-0.307	0.609	1.302	2.219	2.912	3.605
2.1	-0.319	0.592	1.294	2.230	2.942	3.656
2.2	-0.330	0.574	1.284	2.240	2.970	3.705
2.3	-0.341	0.555	1.274	2.248	2.997	3.753
2.4	-0.351	0.537	1.262	2.256	3.023	3.800
2.5	-0.360	0.518	1.250	2.262	3.048	3.845
2.6	-0.368	0.499	1.238	2.267	3.071	3.889
2.7	-0.376	0.479	1.224	2.272	3.093	3.932
2.8	-0.384	0.460	1.210	2.275	3.114	3.973
2.9	-0.390	0.440	1.195	2.277	3.134	4.013
3.0	-0.396	0.420	1.180	2.278	3.152	4.051



Generalized skew coefficients of annual maximum streamflow. (Source: Guidelines for determining flood flow frequency, Bulletin 17B, Hydrology Subcommittee, Interagency Advisory Committee on Water Data, U. S. Geological Survey, Reston, Va. Revised with corrections March 1982.)

Figure 3.5.1

Example 3.5.1

Perform a flood frequency analysis of annual maximum flows from South Fork Wildcat Creek at Lafayette, Indiana.

Step 1: Transform x_i into logarithms, y_i . These are shown in the third column below.

Year _i	x_i (cfs)	y_i
1943	17900	4.2529
1944	5770	3.7612
1945	3380	3.5289
1946	4720	3.6739
1947	3420	3.5340
1948	4080	3.6107
1949	5250	3.7202
1950	12000	4.0792
1951	4250	3.6284
1952	3680	3.5658
1953	5550	3.7443
1954	1530	3.1847
1955	3450	3.5378
1956	1620	3.2095
1957	4250	3.6284
1958	12600	4.1004
1959	8400	3.9243
1960	1820	3.2601
1961	3310	3.5198
1962	5000	3.6990
1963	6420	3.8075
1964	10800	4.0334
1965	3700	3.5682
1966	944	2.9750
1967	5000	3.6990
1968	8550	3.9320
1969	4970	3.6964
1970	3480	3.5416
1971	4040	3.6064
1972	3780	3.5775
1973	2740	3.4378
1974	5740	3.7589
1975	5940	3.7738
1976	4210	3.6243
1977	1740	3.2405
1978	8870	3.9479
1979	3980	3.5999
1980	7160	3.8549
1981	5620	3.7497
1982	4080	3.6107
1983	15100	4.1790
1984	2940	3.4683
1985	6700	3.8261
1986	4740	3.6758
1987	1010	3.0043
1988	3690	3.5670
1989	14400	4.1584
1990	9230	3.9652
1991	8880	3.9484
1992	4100	3.6128

Step 2: Compute the sample statistics of the logarithmic data. There are 50 data points (n=50).

$$\bar{y} = \frac{1}{n} \sum_{i=1}^n y_i = 3.6721$$

$$s_y = \left(\frac{1}{n-1} \sum_{i=1}^n (y_i - \bar{y})^2 \right)^{\frac{1}{2}} = 0.2785$$

$$C_s = \frac{n \sum_{i=1}^n (y_i - \bar{y})^3}{(n-1)(n-2)s_y^3} = -0.2872$$

Step 3: Test for outliers. Referring to Table 3.5.1, for n=50, $K_n=2.768$

$$y_H = \bar{y} + K_n \cdot s_y = 3.6721 + 2.768 \cdot 0.2785 = 4.4430$$

$$Q_H = 10^{4.4430} = 27,732 \text{ cfs}$$

$$y_L = \bar{y} - K_n s_y = 3.6721 - 2.768 \cdot 0.2785 = 2.9013$$

$$Q_L = 10^{2.9013} = 797 \text{ cfs}$$

All of the flows are within the range of 797 and 27,720 cfs. Therefore there are no outliers in this example.

Step 4: Determine the map skew, C_s from Figure 3.5.1.
Lafayette has a map skew of -0.4.

Step 5: Determine the values of G and H. $C_s = -0.2872$.

$$G = -0.33 + 0.08|C_s| = -0.33 + 0.08|-0.2872| = -0.307 \text{ (for } |C_s| \leq 0.9)$$

$$H = 0.94 - 0.26|C_s| = 0.94 - 0.26|-0.2872| = 0.865 \text{ (for } |C_s| \leq 1.50)$$

Step 6: Determine $V(C_s)$, the variance of the station skew.

$$V(C_s) = 10^{G - H \log(n/10)} = 10^{-0.307 - 0.865 \log(50/10)} = 0.1226$$

Step 7: Determine the weight corresponding to station skew, W .

$$W = \frac{0.303}{V(C_s) + 0.303} = \frac{0.303}{0.1226 + 0.303} = 0.712$$

Step 8: Determine the weighted skew of the station.

$$C_w = W \cdot C_s + (1-W) \cdot 0.303 = 0.712 \cdot (-0.2872) + (1-0.712) \cdot 0.303 = -0.117$$

Step 9: Determine the frequency factors, K_T corresponding to the different return periods. Interpolating from Table 3.5.2 between C_w values of -0.1 and -0.2 yields:

<u>Return Period (T_r), years</u>	<u>K_T</u>
2	0.020
5	0.847
10	1.268
25	1.710
50	1.991
100	2.239

Step 10: Determine the flow rates for the various return periods.

$$y_{T_r} = \bar{y} + K_T s_y \quad Q_{T_r} = 10^{y_{T_r}}$$

<u>T_r</u>	<u>K_T</u>	<u>y_{T_r}</u>	<u>Q_{T_r} (cfs)</u>
2	0.020	3.677	4761
5	0.847	3.908	8091
10	1.268	4.025	10,598
25	1.710	4.148	14,071
50	1.991	4.227	16,850
100	2.239	4.296	19,754

3.6 - REGRESSION EQUATIONS

A great deal of research has been done to develop regression equations to generate peak discharges from ungaged rural and urban areas. The USGS (1984) developed equations for the entire nation which were based on selected parameters. The USGS also developed regional regression equations for the State of Indiana dividing the State into the 7 areas shown on Figure 3.6.1. The equations for each region are given below. It can be noted that the parameters used in each equation are different for different regions. The equations are used to determine the peak flowrate associated with return periods of 2, 10, 25, 50, and 100 years. These values can be used in determining the flowrates from larger watersheds for purposes of sizing bridge openings or other hydraulic structures.

Area 1 (16 Stations)

$$Q_2 = 6.72 \text{ DA}^{0.714} (\text{STOR} + 1)^{-0.289} (\text{PREC} - 30)^{0.965}$$

$$Q_{10} = 10.3 \text{ DA}^{0.701} (\text{STOR} + 1)^{-0.262} (\text{PREC} - 30)^{1.060}$$

$$Q_{25} = 11.8 \text{ DA}^{0.697} (\text{STOR} + 1)^{-0.253} (\text{PREC} - 30)^{1.093}$$

$$Q_{50} = 12.9 \text{ DA}^{0.696} (\text{STOR} + 1)^{-0.248} (\text{PREC} - 30)^{1.114}$$

$$Q_{100} = 13.8 \text{ DA}^{0.695} (\text{STOR} + 1)^{-0.243} (\text{PREC} - 30)^{1.132}$$

Area 2 (31 Stations)

$$Q_2 = 26.4 \text{ DA}^{0.708} (\text{STOR} + 1)^{-0.207} \text{RC}^{0.479} (\text{PREC} - 30)^{0.653}$$

$$Q_{10} = 61.8 \text{ DA}^{0.655} (\text{STOR} + 1)^{-0.312} \text{RC}^{0.697} (\text{PREC} - 30)^{0.696}$$

$$Q_{25} = 85.0 \text{ DA}^{0.635} (\text{STOR} + 1)^{-0.357} \text{RC}^{0.782} (\text{PREC} - 30)^{0.702}$$

$$Q_{50} = 106.0 \text{ DA}^{0.619} (\text{STOR} + 1)^{-0.391} \text{RC}^{0.859} (\text{PREC} - 30)^{0.707}$$

$$Q_{100} = 127.0 \text{ DA}^{0.608} (\text{STOR} + 1)^{-0.418} \text{RC}^{0.902} (\text{PREC} - 30)^{0.708}$$

Area 3 (60 Stations)

$$Q_2 = 102 \text{ DA}^{0.758} \text{ SL}^{0.273} (I_{24,2}^{-2.5})^{0.948}$$
$$Q_{10} = 141 \text{ DA}^{0.772} \text{ SL}^{0.384} (I_{24,2}^{-2.5})^{0.894}$$
$$Q_{25} = 158 \text{ DA}^{0.776} \text{ SL}^{0.423} (I_{24,2}^{-2.5})^{0.868}$$
$$Q_{50} = 170 \text{ DA}^{0.777} \text{ SL}^{0.445} (I_{24,2}^{-2.5})^{0.847}$$
$$Q_{100} = 181 \text{ DA}^{0.779} \text{ SL}^{0.466} (I_{24,2}^{-2.5})^{0.831}$$

Area 4 (46 Stations)

$$Q_2 = 16.8 \text{ DA}^{0.435} \text{ SL}^{0.528} L_m^{0.860} (I_{24,2}^{-2.5})^{0.459}$$
$$Q_{10} = 24.1 \text{ DA}^{0.517} \text{ SL}^{0.628} L_m^{0.769} (I_{24,2}^{-2.5})^{0.445}$$
$$Q_{25} = 27.4 \text{ DA}^{0.545} \text{ SL}^{0.664} L_m^{0.741} (I_{24,2}^{-2.5})^{0.448}$$
$$Q_{50} = 29.6 \text{ DA}^{0.554} \text{ SL}^{0.687} L_m^{0.738} (I_{24,2}^{-2.5})^{0.458}$$
$$Q_{100} = 32.0 \text{ DA}^{0.565} \text{ SL}^{0.705} L_m^{0.730} (I_{24,2}^{-2.5})^{0.464}$$

Area 5 (35 Stations)

$$Q_2 = 45.5 \text{ DA}^{0.760} \text{ SL}^{0.390}$$
$$Q_{10} = 67.7 \text{ DA}^{0.780} \text{ SL}^{0.469}$$
$$Q_{25} = 77.0 \text{ DA}^{0.790} \text{ SL}^{0.499}$$
$$Q_{50} = 83.8 \text{ DA}^{0.805} \text{ SL}^{0.516}$$
$$Q_{100} = 91.2 \text{ DA}^{0.811} \text{ SL}^{0.529}$$

Area 6 (32 Stations)

$$Q_2 = 681 \text{ DA}^{0.691} \text{ RC}^{0.856} (I_{24,2} - 2.5)^{1.771}$$

$$Q_{10} = 2177 \text{ DA}^{0.622} \text{ RC}^{0.865} (I_{24,2} - 2.5)^{1.980}$$

$$Q_{25} = 3165 \text{ DA}^{0.598} \text{ RC}^{0.852} (I_{24,2} - 2.5)^{2.035}$$

$$Q_{50} = 3908 \text{ DA}^{0.584} \text{ RC}^{0.849} (I_{24,2} - 2.5)^{2.049}$$

$$Q_{100} = 4734 \text{ DA}^{0.570} \text{ RC}^{0.834} (I_{24,2} - 2.5)^{2.068}$$

Area 7 (22 Stations)

$$Q_2 = 22.6 \text{ DA}^{0.468} \text{ SL}^{0.414} L_m^{0.624} \text{ RC}^{0.846}$$

$$Q_{10} = 45.7 \text{ DA}^{0.350} \text{ SL}^{0.439} L_m^{0.726} \text{ RC}^{0.862}$$

$$Q_{25} = 56.4 \text{ DA}^{0.318} \text{ SL}^{0.458} L_m^{0.754} \text{ RC}^{0.862}$$

$$Q_{50} = 63.6 \text{ DA}^{0.300} \text{ SL}^{0.473} L_m^{0.770} \text{ RC}^{0.860}$$

$$Q_{100} = 70.1 \text{ DA}^{0.285} \text{ SL}^{0.488} L_m^{0.785} \text{ RC}^{0.854}$$

In these equations DA is the contributing drainage area (mi²), SL is the slope (feet per mile), L is the length (miles), STOR is the storage in percent (a value of 1.00 was added to each value of STOR used in regression), PREC is the mean annual precipitation in inches (a value of 30.00 was subtracted from each value of PREC used in regression), RC is the soil runoff coefficient, in inches per inch, and $I_{24,2}$ is the rainfall intensity, in inches for the 2-year, 24-hour storm (a value of 2.50 was subtracted from each value of $I_{24,2}$ used in regression). The mean annual precipitation values are found on Figure 3.6.2.

Example 3.6.1 illustrates the use of these equations.

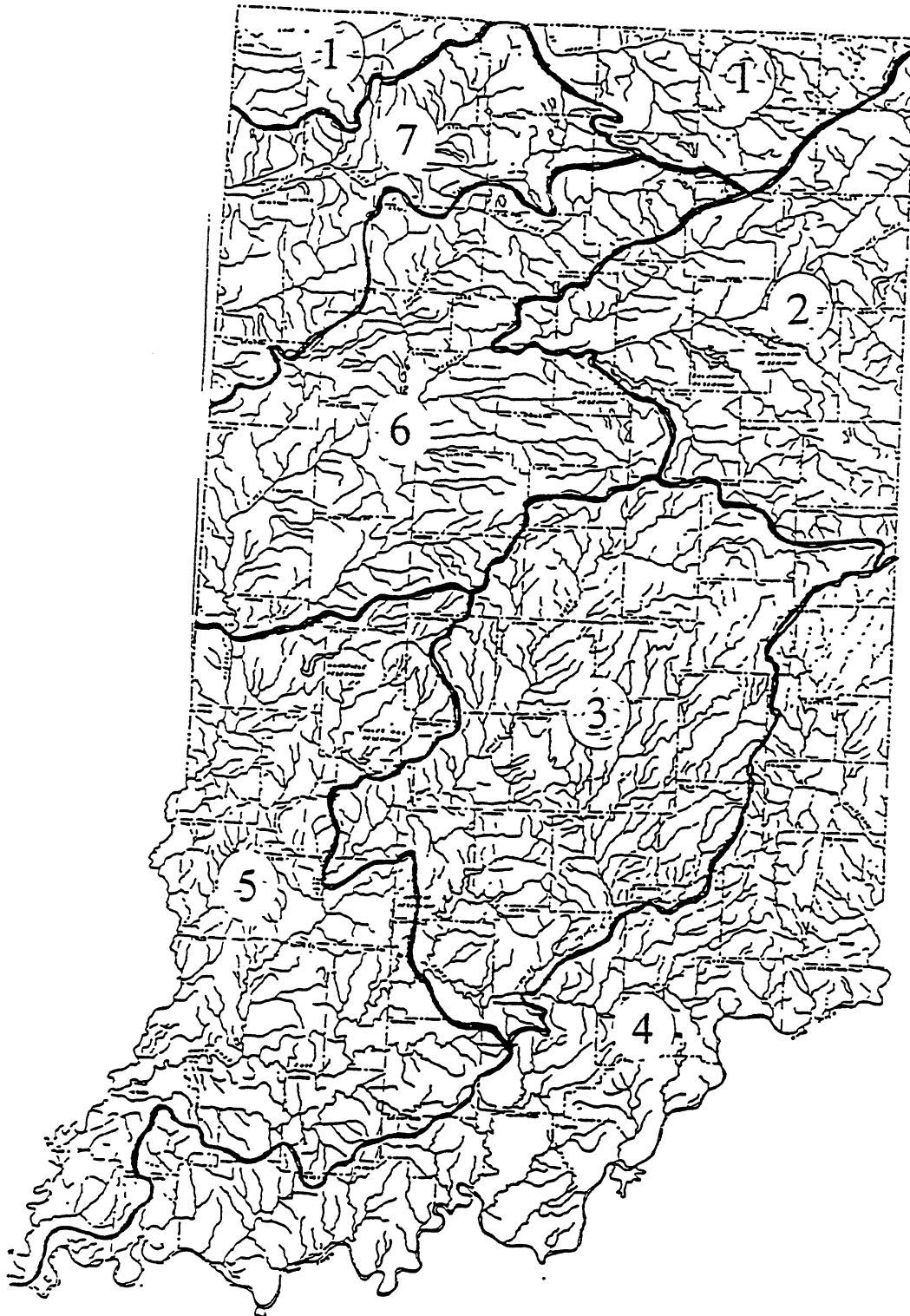


Figure 3.6.1
Location of Areas Used in Regression Study

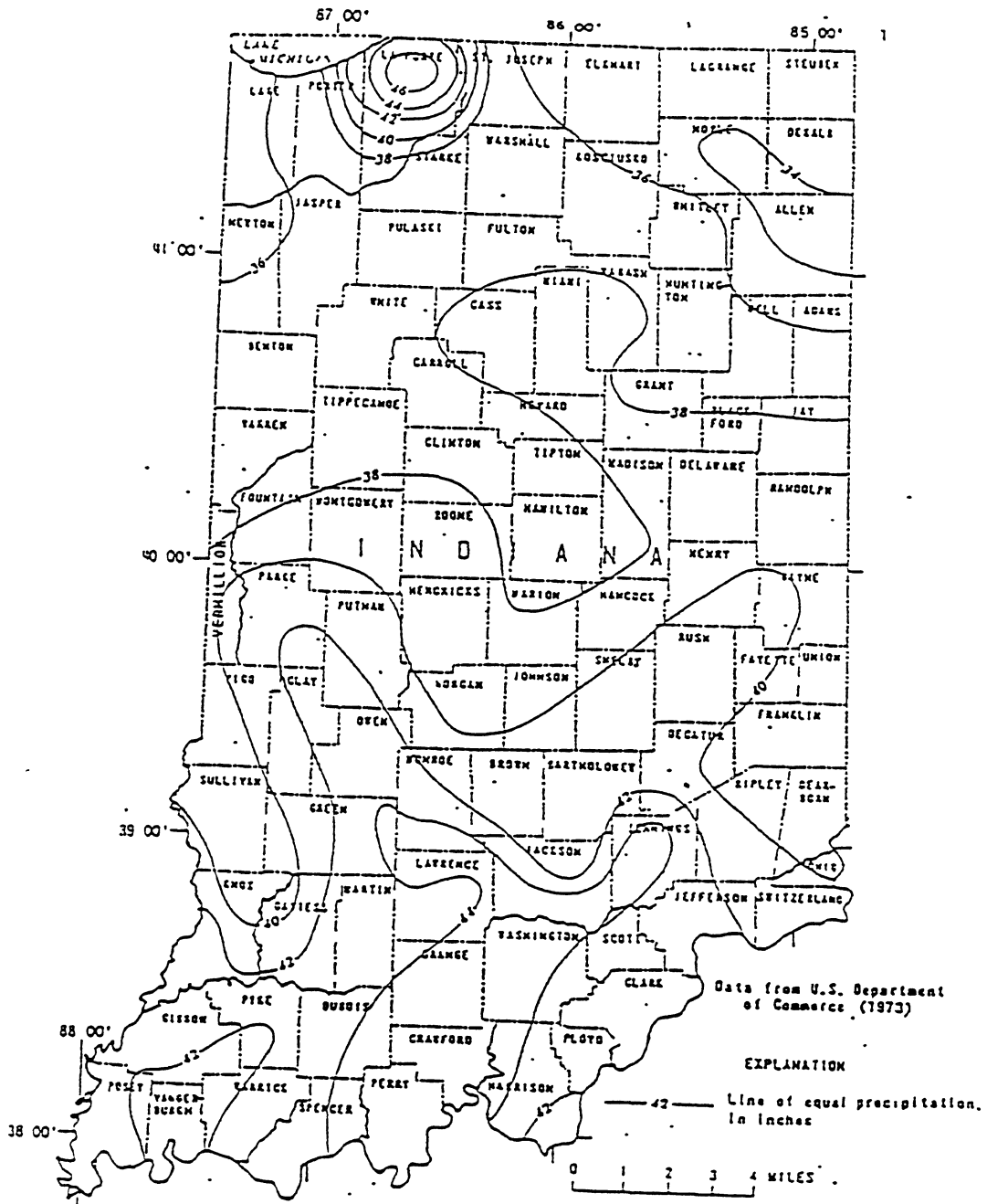


Figure 3.6.2
Mean Annual Precipitation in Indiana 1940 - 1970

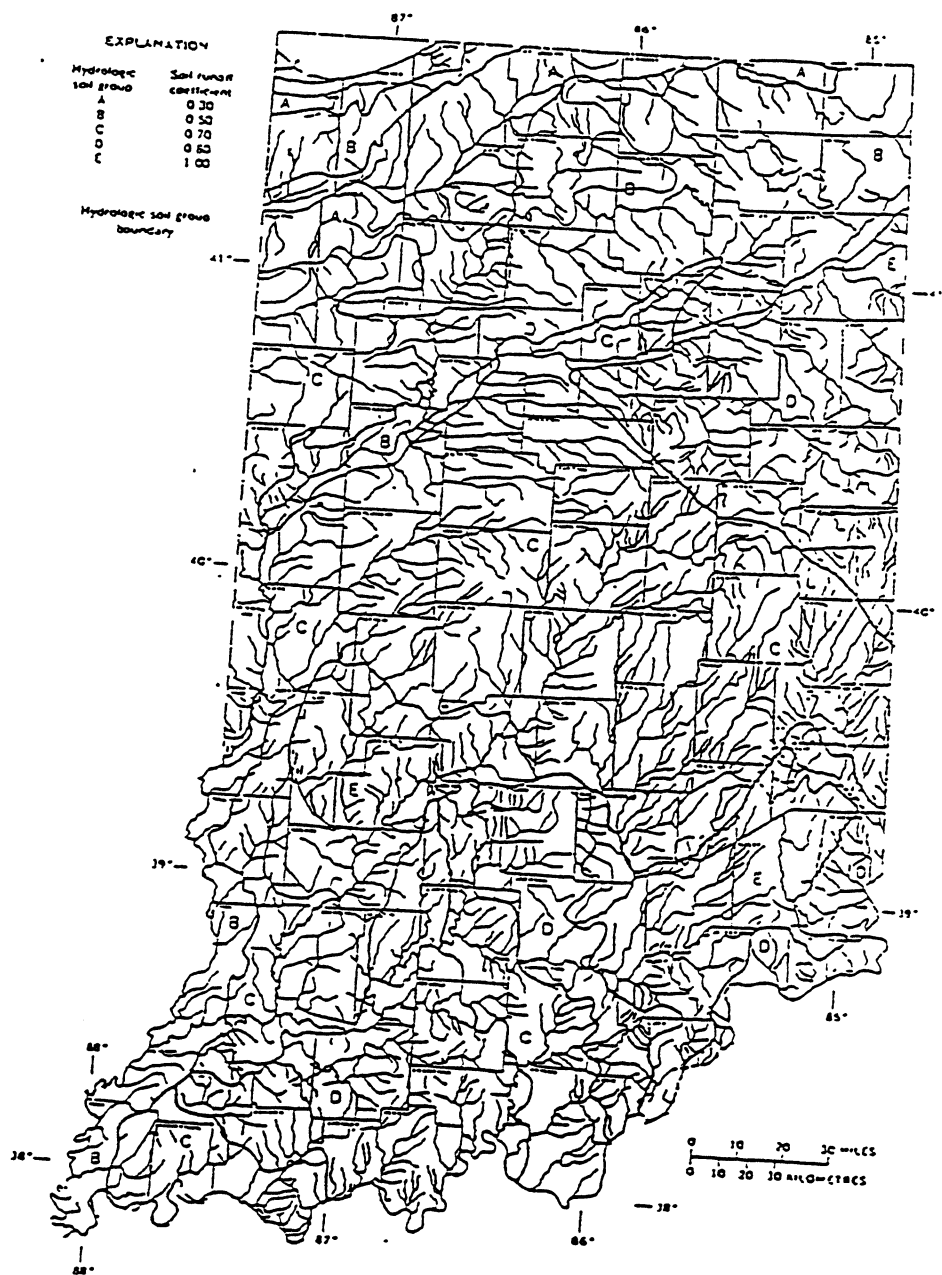


Figure 3.6.3
Major Hydrologic Soil Groups in Indiana

Example 3.6.1

This example illustrates the use of the regional regression equations

Find the peak runoff for a 10-year and 100-year event for the 243 square mile watershed of the South Fork Wildcat Creek at Lafayette, Indiana. Assume a runoff coefficient of 0.7.

From Figure 3.6.2, the mean annual precipitation for this area is approximately 37.5 inches. In order to find Q_{10} and Q_{100} the equations require the 2-year 24-hour rainfall intensity for the Lafayette area. Using Chen's method (described in Section 2.2.5) the intensity can be found as follows:

$$\begin{aligned} p_1^{10} &= 1.95" & p_{24}^{10} &= 4.18" & p_1^{100} &= 2.72" \\ (p_1^{10}) / (p_{24}^{10}) &= 1.95" / 4.18" = 0.4665 \\ X &= (p_1^{100}) / (p_1^{10}) = 2.72" / 1.95" = 1.395 \end{aligned}$$

$$\text{From Figure 2.2.3} \quad a_1 = 28.6 \quad d' = 9.2 \quad n' = 0.795$$

$$\begin{aligned} i_1^{10} &= p_1^{10} / 1 \text{ hr} = 1.95"/\text{hr} \\ a' &= a_1 i_1^{10} \log(10^{(2-X)} T_r^{(X-1)}) = 55.77 \log(4.03 T_r^{(0.395)}) \end{aligned}$$

$$i_{24}^2 = \frac{a'}{(60t + d')^{n'}} = \frac{55.77 \log(4.03 (2)^{0.395})}{(1440 + 9.2)^{0.795}} = 0.124 \text{ inches/hour}$$

$$I_{24,2} = 0.124 \text{ in/hr} * 24 \text{ hr} = 2.98 \text{ inches}$$

Using the regression equations for area 6:

$$\begin{aligned} Q_{10} &= 2177 DA^{0.622} RC^{0.865} (I_{24,2} - 2.5)^{1.980} \\ &= 2177 (243)^{0.622} (0.7)^{0.865} (2.98 - 2.5)^{1.980} \\ &= 11,391 \text{ cfs} \end{aligned}$$

$$\begin{aligned} Q_{100} &= 4734 DA^{0.570} RC^{0.834} (I_{24,2} - 2.5)^{2.068} \\ &= 4734 (243)^{0.570} (0.7)^{0.834} (2.98 - 2.5)^{2.068} \\ &= 17,646 \text{ cfs} \end{aligned}$$

3.7 - COORDINATED DISCHARGES

The Corps of Engineers, USGS, SCS and Indiana Department of Natural Resources (IDNR) have coordinated the discharges of selected streams in Indiana (1990). This publication consists of discharge vs. drainage area curves which have been developed using available stream gage data. These data were statistically analyzed for the 10, 25, 50, and 100-year events and the peak flowrates at selected points (with a known drainage area) were plotted on log-log paper. A typical curve is shown on Figure 3.7.1.

This information should be used when performing Flood Insurance Studies or planning flood control projects and are useful for larger areas. For more information, contact the IDNR at the address at the end of the Reference section.

Example 3.7.1

Find the 10-year and 100-year flowrate for the South Fork Wildcat Creek at Lafayette using Figure 3.7.1 with a drainage area of 243 square miles.

From this exhibit the 10-year flowrate is approximately 9300 cfs and the 100-year flowrate is approximately 17,600 cfs.

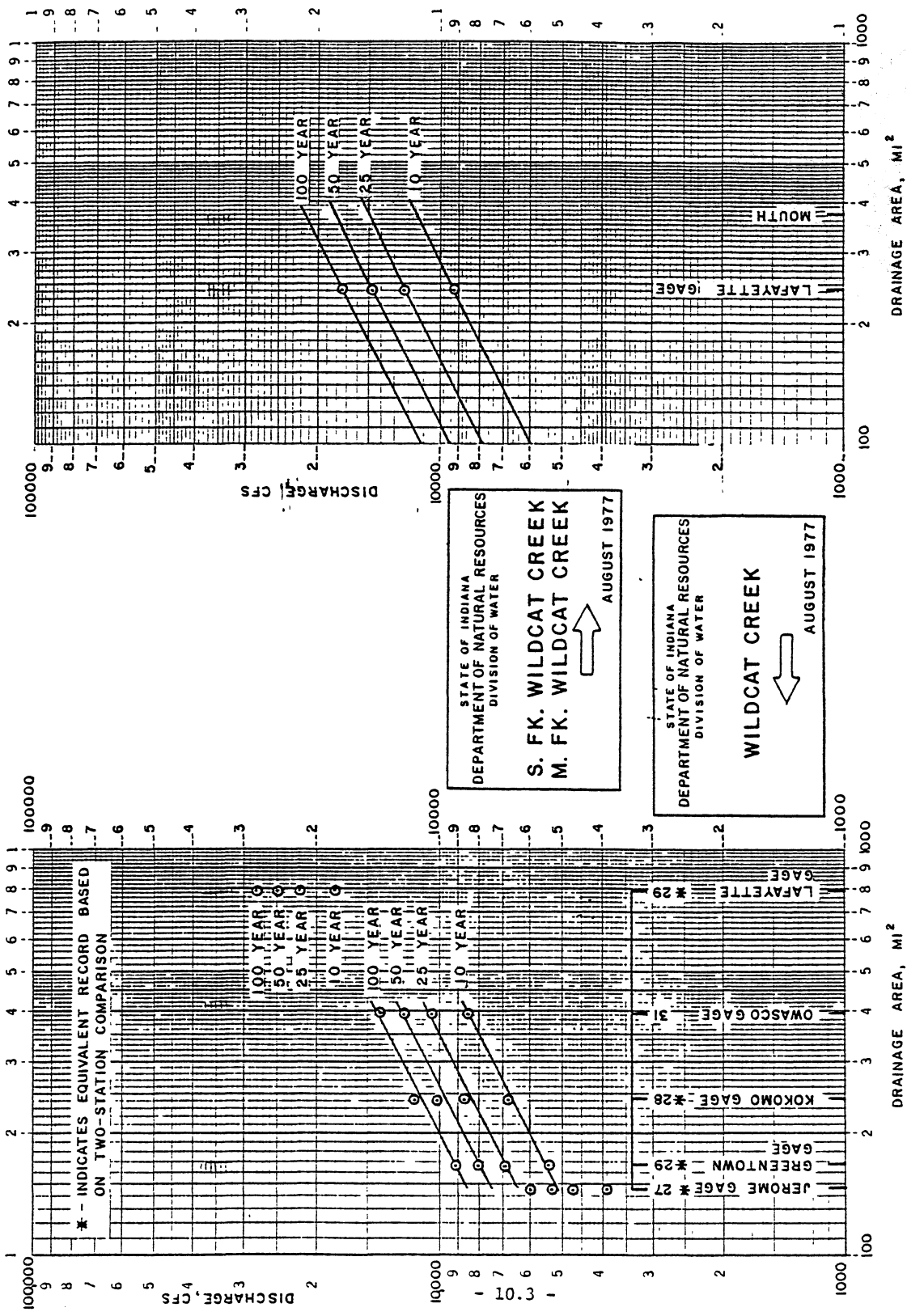


Figure 3.7.1
Coordinated Discharge Curve, Example 3.7.1 (Indiana DNR, 1990)

3.8 - FLOOD INSURANCE STUDIES

Flood Insurance Studies (FIS) have been performed by the Federal Emergency Management Agency (FEMA) for selected areas throughout Indiana to determine flood elevations for major waterways. As part of these studies discharges were calculated for various frequencies. These studies should be consulted when discharges are needed. For information on the areas where studies have been performed the FEMA maps can be obtained from the Indiana Department of Natural Resources, Division of Water. The current address is located at the end of the References.

Example 3.7.1

Find the 10-year and 100-year flowrate for the 243 mi² drainage area of the South Fork Wildcat Creek at Lafayette using the Flood Insurance Study by FEMA for Tippecanoe County, Unincorporated Areas.

A copy of the study is not included. From the tables at the beginning of the report the 10-year flowrate is 9,200 cfs and the 100-year flowrate is 17,500 cfs for the South Fork Wildcat Creek at Lafayette.

The following table compares the results of Examples 3.5.1, 3.6.1, 3.7.1 and 3.8.1 which demonstrate four different approaches to finding the runoff from a drainage area. The values show fairly close results. The difference between the 100-year peak discharge computed by the Water Resource Council Method and the other methods is because the WRC Method calculations include the most recent yearly data that have some very high peak flows within the last 15 years. When the WRC Method is computed for the same number of years as the other methods the 10-year flowrate is 9,692 cfs and the 100-year flowrate is 18,652 cfs.

Table 3.8.1
Comparison of Results from Examples 3.5.1, 3.6.1, 3.7.1 and 3.8.1

Recurrence Interval	WRC Method	Regression Equations	Coordinated Discharges	Flood Insurance Study
10-Year	10,598 cfs	11,391 cfs	9,300 cfs	9,200 cfs
100-Year	19,754 cfs	17,646 cfs	17,600 cfs	17,500 cfs

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Coordinated Discharges of selected streams and Flood Insurance Rate maps can be obtained by writing to:

Department of Natural Resources, Division of Water
402 W. Washington St., Room W264
Indianapolis, Indiana 46204
(317) 232-4162

Chapter 4

OPEN CHANNELS

Chapter 4 - OPEN CHANNELS

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Chapter 4 - OPEN CHANNELS

LIST OF PARAMETERS

A	Cross-sectional area (ft^2 or m^2)
b	Bottom width of channel
B_s	Surface width of the channel
d	Depth of flow (normal to flow) (ft or m)
d_{max}	Maximum permissible depth of flow
D	Hydraulic depth (ft or m)
E	Specific energy (ft or m)
ΔE	Change in specific energy
F	Froude number
g	Acceleration constant due to gravity (ft/sec^2 or m/sec^2)
γ	Specific weight of water (lb/ft^3 or N/m^3)
h_L	Head loss (ft or m)
K	Ratio of the tractive force
K'	Conveyance
K_b	Maximum shear on the bottom (lb/ft^2)
K_s	Maximum shear on the sides (lb/ft^2)
L	Length (ft or m)
n	Manning's roughness coefficient
n_E	Equivalent roughness coefficient
n_j	Roughness coefficient for a subsection
P	Pressure
P_w	Wetted perimeter
ϕ	Angle of the channel side-slope
Q	Volumetric flowrate (ft^3/sec or m^3/sec)
R	Hydraulic radius
R_o	Mean radius of the bend
S	Slope of the energy grade line (ft/ft or m/m)
SF	Section Factor
S_o	Channel bed slope (ft/ft or m/m)
S_w	Water surface slope
T	Top width of channel
τ_b	Tractive force of channel bed (lb/ft^2 or
τ_o	Average bed shear force, tractive force (lb/ft^2 or
τ_s	Tractive force of channel side (lb/ft^2 or
θ	Angle of repose of soil
Δt	Time increment
V	Velocity (ft/sec or m/sec)
ΔV	Change in velocity (ft/sec or m/sec)
X	Composite length (ft or m)
ΔX	Increment of length (ft or m)

LIST OF PARAMETERS (cont'd)

y	Depth of flow (vertically) (ft or m)
y_c	Critical depth
y_n	Normal depth (ft or m)
z	Elevation from datum (ft or m)
z	Side-slope

Chapter 4 - OPEN CHANNELS

4.1 - INTRODUCTION

An open channel is a conveyance in which the liquid stream is not completely enclosed by solid boundaries. Thus the stream has a free surface subjected only to atmospheric pressure. Since the pressure is zero (gage pressure) at the surface, the energy available to cause fluid flow is due to changes in elevation from one section to another in the channel. The driving force for the flow is the component of the liquid weight along the slope of the channel. This driving force is resisted by a shear force transmitted from the boundaries to the liquid. The principle types of open channels are natural rivers and streams, artificial canals, drainage ditches, sewers, tunnels, and pipelines which are not completely filled.

The accurate solution of flow problems in open channels is difficult. Reliable experimental data are difficult to secure and there is a wide range of possible flow conditions. The cross-section of open channels may be of any shape, from circular to the irregular forms of natural streams. The channel surfaces vary greatly from concrete-lined channels to boulders, trees and other vegetation which occur in natural channels. In addition, the bed of the channel may be changing with time as erosion and deposition of bed materials occur during changing flow conditions.

The geometry and classification of open channels and the methods of flow computation, analysis and design are discussed in this chapter.

4.1.1 - Channel Geometry

A few definitions and relationships will be useful in the discussion of open channel flow. The term channel section refers to the cross-section of the channel normal to the direction of flow. Natural channel sections are irregular. Artificial channels (man-made) are usually designed with sections of a regular geometric shape. When these sections are regular, geometric properties of the section can be easily derived, based on the properties of the section and the depth of flow. If the section is irregular, these elements can be determined graphically.

The depth of flow, y , is the vertical distance from the liquid surface to the lowest point of the channel section. This depth of flow is often used interchangeably with the depth of the flow section, d . Strictly speaking, d is measured normal to the flow while y is measured vertically. It is only in channels with a steep longitudinal slope that there is a significant difference in the two values as seen in Figure 4.1.1.

The top width, T , is the width of the channel section at the free surface. The water area, A , is the cross-sectional area of the flow normal to the direction of flow. The wetted perimeter,

P_w is the length of the solid channel boundary in contact with the fluid in a given cross-section. The hydraulic radius, R , is the area divided by the wetter perimeter ($R = A/P_w$). The hydraulic depth, D , is the area divided by the top width ($D = A/T$). Finally, the section factor, SF which will be used later, is the area times the square root of the hydraulic depth ($SF = A\sqrt{D}$). Table 4.1.1 presents all the geometric elements for some commonly occurring sections.

4.1.2 - Classification of Flow in Open Channels

Since flow in open channels involves a free surface, it has many degrees of freedom. The classifications for these conditions include uniform and non-uniform (varied) flow; steady or unsteady flow; and supercritical (rapid) or subcritical flow (tranquil).

Uniform flow exists when the depth of flow does not change throughout a reach with a constant slope and cross section. Obviously, this condition rarely occurs in nature. For uniform flow to exist, the drop in potential energy due to the fall in elevation along the channel must be consumed, exactly, by the energy dissipation due to boundary friction and turbulence.

Non-uniform flow occurs when there is a change in depth due to a slope change, barrier or drop or a change in the cross-section so that the velocity increases or decreases in the direction of the flow. This type of flow is termed varied flow -- gradually varied flow if changes occur slowly, and rapidly varied if large changes occur in short distances in the channel.

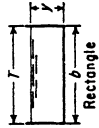
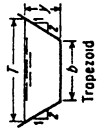
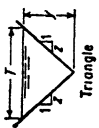
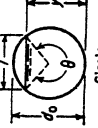
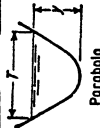

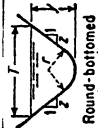
Steady flow occurs when the velocity at a point does not change with time -- that is $\Delta V/\Delta t = 0$. When the flow is unsteady, $\Delta V/\Delta t \neq 0$. Unsteady flows are difficult to analyze unless the change with time is sufficiently slow to permit a step type of analysis. Examples of unsteady flow are traveling surges and flood waves in an open channel.

The classification of flows as subcritical and supercritical flow is based on the Froude number ($F = V/\sqrt{gD}$), which is the ratio of the inertial force to that of gravity. If $F < 1.0$ the flow is subcritical (tranquil) flow, $F = 1.0$ it is critical flow and $F > 1.0$ it is supercritical (rapid) flow.

In summary, flow in open channels may be broken down into three classifications:

1. uniform or non-uniform
2. steady or unsteady
3. subcritical or supercritical

Table 4.1.1
Geometric Elements of Channel Sections (Chow, 1988)

Section	Area A	Wetted perimeter P_w	Hydraulic radius R	Top width T	Hydraulic depth D	Section factor S_F
 Rectangle	by	$b + 2y$	$\frac{by}{b + 2y}$	b	y	$by^{1.5}$
 Trapezoid	$(b + zy)y$	$b + 2y\sqrt{1 + z^2}$	$\frac{(b + zy)y}{b + 2y\sqrt{1 + z^2}}$	$b + 2zy$	$\frac{(b + zy)y}{b + 2zy}$	$\frac{[(b + zy)y]^{1.5}}{\sqrt{b + 2zy}}$
 Triangle	zy^2	$2y\sqrt{1 + z^2}$	$\frac{zy}{2\sqrt{1 + z^2}}$	$2zy$	$\frac{1}{2}y$	$\frac{\sqrt{2}}{2}zy^{2.5}$
 Circle	$\frac{1}{8}(\theta - \sin \theta)d_0^3$	$\frac{1}{2}\theta d_0$	$\frac{1}{4}\left(1 - \frac{\sin \theta}{\theta}\right)d_0$	$\frac{(\sin \frac{1}{2}\theta)d_0}{2\sqrt{y(d_0 - y)}}$ or $\frac{(\sin \frac{1}{2}\theta)d_0}{2\sqrt{y(d_0 - y)}}$	$\frac{1}{8}\left(\frac{\theta - \sin \theta}{\sin \frac{1}{2}\theta}\right)d_0$	$\frac{\sqrt{2}}{32}\frac{(\theta - \sin \theta)^{1.5}}{(\sin \frac{1}{2}\theta)^{0.5}}d_0^{2.5}$
 Parabola	$\frac{2}{3}Ty$	$T + \frac{8}{3}\frac{y^2}{T}$	$\frac{2Ty^2}{3T^2 + 8y^2}$	$\frac{3A}{2y}$	$\frac{3}{2}y$	$\frac{3}{2}\sqrt{6}Ty^{1.5}$
 Round-cornered rectangle ($y > r$)	$\left(\frac{\pi}{2} - 2\right)r^2 + (b + 2r)y$	$(\pi - 2)r + b + 2y$	$\frac{(\pi/2 - 2)r^2 + (b + 2r)y}{(\pi - 2)r + b + 2y}$	$b + 2r$	$\frac{(\pi/2 - 2)r^2}{b + 2r} + y$	$\frac{[(\pi/2 - 2)r^2 + (b + 2r)y]^{1.5}}{\sqrt{b + 2r}}$
 Round-bottomed triangle	$\frac{T^2}{4z} - \frac{r^2}{z}(1 - z \cot^{-1} z)$	$\frac{T}{z}\sqrt{1 + z^2} - \frac{2r}{z}(1 - z \cot^{-1} z)$	$\frac{A}{P}$	$2[z(y - r) + r\sqrt{1 + z^2}]$	$\frac{A}{T}$	$A\sqrt{\frac{A}{T}}$

* Satisfactory approximation for the interval $0 < z \leq 1$, where $z = 4y/T$. When $z > 1$, use the exact expression $P = (T/2)[\sqrt{1 + z^2} + 1/z \ln(z + \sqrt{1 + z^2})]$.

4.1.3 - Energy Equation in Open Channels

In Figure 4.1.1 the geometric elements of open channel flow discussed in Section 4.1.1 are shown. If the energy equation is written between points 1 and 2 along the channel, the following relationship is found,

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + z_2 + h_L \quad (4.1.1)$$

where h_L = head loss through reach (ft) and γ = specific weight of water (lbs/ft³).

Since atmospheric pressure acts on the free surface, the pressure terms become zero (gage pressure), and if the distance above the datum is written as $(z + y)$, Equation 4.1.1 becomes:

$$\frac{V_1^2}{2g} + z_1 + y_1 = \frac{V_2^2}{2g} + z_2 + y_2 + h_L \quad (4.1.2)$$

For small bed slopes (S_o) the values of z_1 and z_2 are essentially equal, which gives Equation 4.1.3

$$\frac{V_1^2}{2g} + y_1 = \frac{V_2^2}{2g} + y_2 + h_L \quad (4.1.3)$$

In Figure 4.1.1, S_w is defined as the water surface slope which for small slopes may be written as

$$S_w = \frac{y_1 - y_2}{L} \quad (4.1.4)$$

The slope of the energy grade line is observed to be

$$S = h_L/L \quad (4.1.5)$$

In uniform flow, the water surface slope (S_w), the energy slope (S) and the bed slope (S_o) are all equal to one another

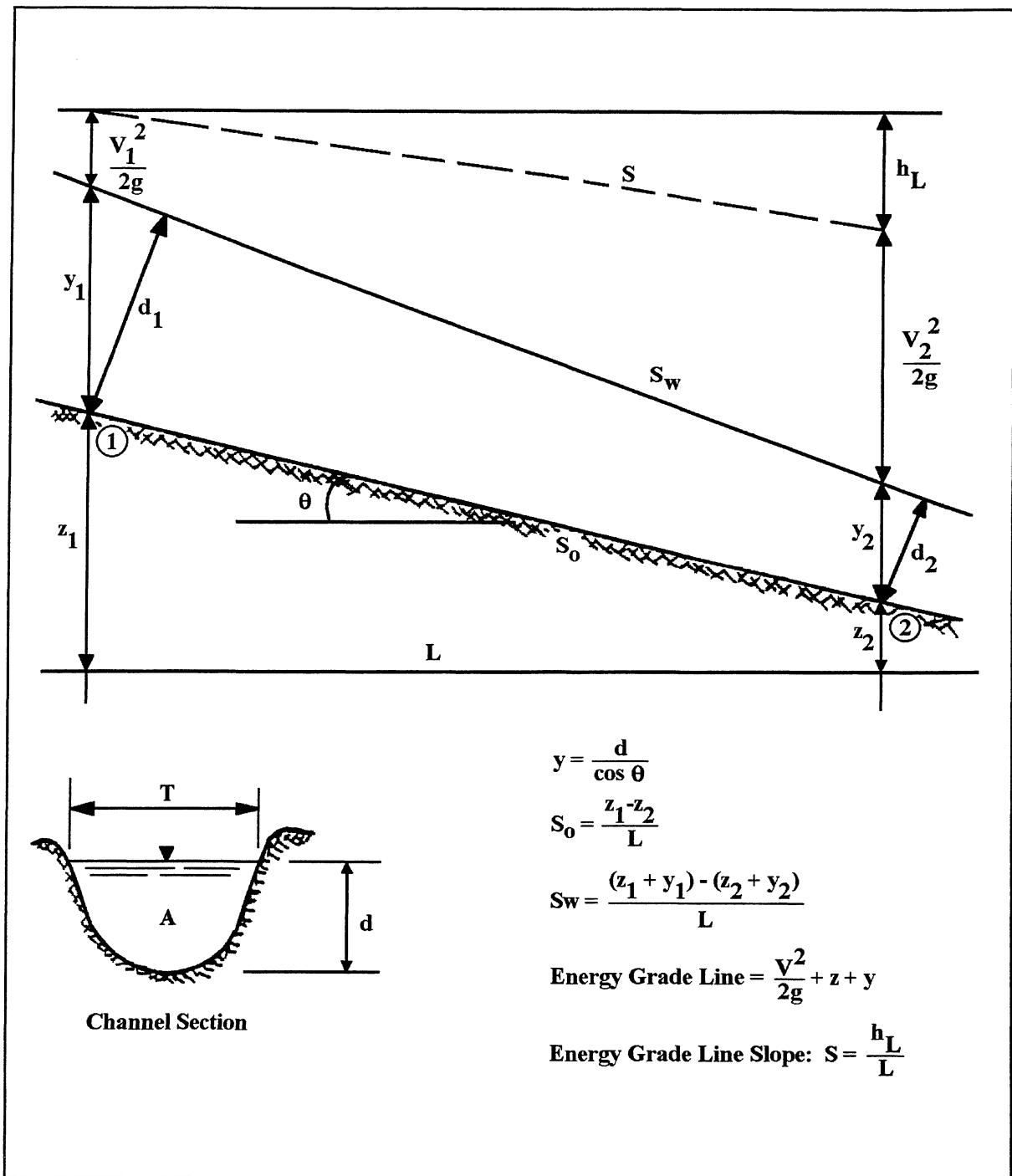


Figure 4.1.1
Properties of Open Channel Flow

4.2 - EQUATIONS FOR UNIFORM FLOW

One of the most widely used formulas for open channel flow is the Manning Equation. In English units, the formula is written as,

$$Q = \frac{1.49}{n} (R)^{2/3} A(S)^{1/2} \quad (4.2.1)$$

where Q = volumetric flowrate (cfs), R = hydraulic radius (ft), A = cross-sectional area (ft²), S = channel slope (ft/ft) or slope of the energy grade line (ft/ft), n = Manning's roughness coefficient.

(A nomograph solution of Equation 4.2.1 for circular pipe is given in Figure 4.4.3)

The same equation may be used with System International Units if the 1.49 factor is omitted. Table 4.2.1 includes various roughness factors for natural and man-made open channels.

It should be emphasized that strictly speaking, the Manning Equation applies only to uniform flow of water ($S = S_o = S_w$); although as will be shown later, it may also be applied to slowly varied non-uniform flow. Examples 4.2.1 and 4.2.2 help to illustrate the use of this equation.

When the Manning Equation is used for open channels, the computations involve the following six (6) variables:

1. The normal discharge, Q
2. The mean velocity of flow, V
3. The normal depth, y_n , associated with steady, uniform flow
4. The channel roughness coefficient, n
5. The channel slope, S
6. The geometric elements which depend upon the shape of the channel section, such as A , R , etc.

When any four of the above variables are given, the remaining two may be obtained using Equation 4.2.1 and the continuity equation, $Q = AV$. The following is a list of some of the possible problems in flow computation.

- (a) The computation of the normal discharge, Q . In practical applications, this computation is required to determine the capacity of the channel section.
- (b) To determine the velocity. This velocity is often required for the study of scouring and silting in channels.
- (c) To compute the normal depth. This computation is necessary for the determination of the stage of flow in a given channel.

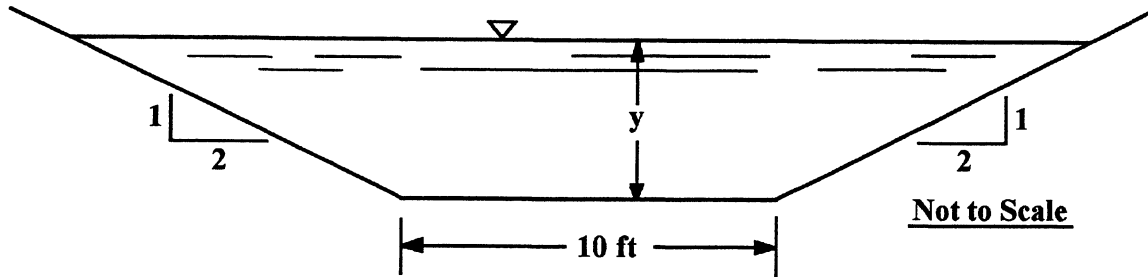
Table 4.2.1
Values of Manning's Roughness Coefficient, n
(Federal Highway Administration, 1973)

	Manning's a range ¹		Manning's a range ¹
I. Cleared conduits:			
A. Concrete pipe.....	0.011-0.013	IV. Highway channels and swales with maintained vegetation^{1,2}	
(value shown are for velocities of 2 and 6 f.p.s.):		A. Depth of flow up to 0.7 foot:	
B. Corrugated-metal pipe or pipe-arch:		1. Bermudagrass, Kentucky bluegrass, buffalograss:	0.07-0.045
1. 2½ by ½-in. corrugation (riveted pipe): ³	0.024	a. Mowed to 2 inches.....	0.09-0.08
a. Plain or fully coated.....		b. Length 4-6 inches.....	0.12-0.09
b. Paved invert (range values are for 25 and 50 percent		2. Good stand, any grass:	0.20-0.13
of circumference paved):		a. Length about 12 inches.....	0.14-0.08
(1) Flow full depth.....	0.021-0.018	b. Length about 24 inches.....	0.25-0.13
(2) Flow 0.8 depth.....	0.021-0.016	3. Fair stand, any grass:	
(3) Flow 0.6 depth.....	0.019-0.013	a. Length about 12 inches.....	0.14-0.08
2. 6 by 2-in. corrugation (field bolted).....	0.03	b. Length about 24 inches.....	0.25-0.13
C. Vitrified clay pipe.....	0.012-0.014	B. Depth of flow 0.7-1.5 feet:	
D. Cast-iron pipe, uncoated.....	0.013	1. Bermudagrass, Kentucky bluegrass, buffalograss:	0.05-0.035
E. Steel pipe.....	0.009-0.011	a. Mowed to 2 inches.....	0.06-0.04
F. Brick.....	0.014-0.017	b. Length 4 to 6 inches.....	0.13-0.07
G. Monolithic concrete:		2. Good stand, any grass:	0.20-0.16
1. Wood forms, rough.....	0.015-0.017	a. Length about 12 inches.....	0.14-0.08
2. Wood forms, smooth.....	0.012-0.014	b. Length about 24 inches.....	0.17-0.08
3. Steel forms.....	0.012-0.013	3. Fair stand, any grass:	
H. Cemented rubble masonry walls:		a. Length about 12 inches.....	0.10-0.06
1. Concrete floor and top.....	0.017-0.022	b. Length about 24 inches.....	0.17-0.08
2. Natural floor.....	0.019-0.025	V. Street and expressway gutters:	
I. Laminated treated wood.....	0.015-0.017	A. Concrete gutter, troweled finish.....	0.012
J. Vitrified clay liner plates.....	0.015	B. Asphalt pavement:	
II. Open channels, lined⁴ (straight alignment):⁵		1. Smooth texture.....	0.013
A. Concrete, with surfaces as indicated:		2. Rough texture.....	0.016
1. Formed, no finish.....	0.012-0.017	C. Concrete gutter with asphalt pavement:	
2. Trowel finish.....	0.012-0.014	1. Smooth.....	0.013
3. Float finish.....	0.013-0.016	2. Rough.....	0.016
4. Float finish, some gravel on bottom.....	0.015-0.017	D. Concrete pavement:	
5. Unite, good section.....	0.016-0.019	1. Float finish.....	0.014
6. Unite, wavy section.....	0.018-0.022	2. Broom finish.....	0.016
B. Concrete, bottom float finished, sides as indicated:		E. For gutters with small slope, where sediment may accu-	
1. Dressed stone in mortar.....	0.015-0.017	ulate, increase above values of a by.....	0.005
2. Random stone in mortar.....	0.017-0.020	VI. Natural stream channels:⁶	
3. Cement rubble masonry.....	0.020-0.025	A. Minor streams ⁷ (surface width at flood stage less than 100	
4. Cement rubble masonry, plastered.....	0.016-0.020	ft.):	
5. Dry rubble (riprap).....	0.020-0.030	1. Fairly regular section:	
C. Gravel bottom, sides as indicated:		a. Some grass and weeds, little or no brush.....	0.030-0.035
1. Formed concrete.....	0.017-0.020	b. Dense growth of weeds, depth of flow materially	
2. Random stone in mortar.....	0.020-0.022	greater than weed height.....	0.035-0.06
3. Dry rubble (riprap).....	0.023-0.033	c. Some weeds, light brush on banks.....	0.035-0.06
D. Brick.....	0.014-0.017	d. Some weeds, heavy brush on banks.....	0.05-0.07
E. Asphalt:		e. Some weeds, dense willows on banks.....	0.06-0.08
1. Smooth.....	0.012	f. For trees within channel, with branches submerged	
2. Rough.....	0.016	at high stage, increase all above values by.....	0.01-0.02
F. Wood, planed, clean.....	0.011-0.013	2. Irregular sections, with pools, slight channel meander;	
G. Concrete-lined excavated rock:		increase values given in 1a-c about.....	0.01-0.02
1. Good section.....	0.017-0.020	3. Mountain streams, no vegetation in channel, banks	
2. Irregular section.....	0.023-0.027	usually steep, trees and brush along banks sub-	
III. Open channels, excavated⁴ (straight alignment,⁸ natural		merged at high stage:	
lining):		a. Bottom of gravel, cobbles, and few boulders.....	0.04-0.06
A. Earth, uniform section:		b. Bottom of cobbles, with large boulders.....	0.06-0.07
1. Clean, recently completed.....	0.016-0.018	B. Flood plains (adjacent to natural streams):	
2. Clean, after weathering.....	0.018-0.020	1. Pasture, no brush:	
3. With short grass, few weeds.....	0.022-0.027	a. Short grass.....	0.030-0.025
4. In gravelly soil, uniform section, clean.....	0.022-0.026	b. High grass.....	0.035-0.06
B. Earth, fairly uniform section:		2. Cultivated areas:	
1. No vegetation.....	0.022-0.026	a. No crop.....	0.03-0.04
2. Grass, some weeds.....	0.025-0.030	b. Mature row crops.....	0.035-0.045
3. Dense weeds or aquatic plants in deep channels.....	0.030-0.034	c. Mature field crops.....	0.04-0.06
4. Sides clean, gravel bottom.....	0.025-0.030	2. Heavy weeds, scattered brush.....	0.05-0.07
5. Sides clean, cobble bottom.....	0.030-0.040	3. Light brush and trees: ⁹	
C. Dracine excavated or dredged:		a. Winter.....	0.05-0.06
1. No vegetation.....	0.028-0.033	b. Summer.....	0.06-0.08
2. Light brush on banks.....	0.033-0.040	5. Medium to dense brush: ¹⁰	
D. Rock:		a. Winter.....	0.07-0.11
1. Based on design section.....	0.035	b. Summer.....	0.10-0.16
2. Based on actual mean section:		6. Dense willows, summer, not bent over by current.....	0.15-0.20
a. Smooth and uniform.....	0.035-0.040	7. Cleared land with tree stumps, 100-150 per acre:	
b. Jagged and irregular.....	0.040-0.045	a. No sprouts.....	0.04-0.05
E. Channels not maintained, weeds and brush most:		b. With heavy growth of sprouts.....	0.06-0.08
1. Dense weeds, high as flow depth.....	0.06-0.12	8. Heavy stand of timber, a few down trees, little under-	
2. Clean bottom, brush on sides.....	0.05-0.08	growth:	
3. Clean bottom, brush on sides, highest stage of flow.....	0.07-0.11	a. Flood depth below branches.....	0.10-0.12
4. Dense brush, high stage.....	0.10-0.14	b. Flood depth reaches branches.....	0.12-0.16
Major streams (surface width at flood stage more than			
100 ft.): Roughness coefficient is usually less than for			
minor streams of similar description on account of less			
effective resistance offered by irregular banks or vege-			
tation on banks. Values of a may be somewhat re-			
duced. Follow recommendation in publication cited ¹¹			
if possible. The value of a for larger streams of most			
regular section, with no boulders or brush, may be in the			
range of.....			

- (d) The determination of the channel roughness. This computation is used to ascertain the roughness coefficient in a given channel which can then be used in similar channels.
- (e) The computation of the channel slope. This is needed for the adjustment of the slope in a given channel.
- (f) The determination of the channel dimensions. This is needed in the design of open channels.

Example 4.2.1

Determine the normal depth for uniform flow in the trapezoidal channel shown below with a flowrate of 225 cfs, a slope of 0.0007 ft/ft and a Manning n of 0.015



From Table 4.1.1,

$$A = (b + zy)y$$

$$P_w = b + 2y\sqrt{1 + z^2}$$

$$R = A/P$$

$$A = (10 + 2y)y$$

$$P_w = 10 + 2y\sqrt{1 + 2^2} = 10 + 2y\sqrt{5}$$

$$R = \frac{(10 + 2y)y}{10 + 2y\sqrt{5}}$$

Hence,

$$Q = 225 = \frac{1.49}{0.015} (10 + 2y)y \left[\frac{(10 + 2y)y}{10 + 2y\sqrt{5}} \right]^{2/3} (0.0007)^{1/2}$$

or

$$225 = 2.63 (10 + 2y)y \left[\frac{(10 + 2y)y}{10 + 2y\sqrt{5}} \right]^{2/3}$$

solving by trial and error gives a normal depth $y_n = y = 3.16$ ft.

Example 4.2.2

Determine the normal discharge for a trapezoidal channel with a bottom width of 10 feet and side slopes of 2.5 on 1. The depth of flow is 5.4 ft. and the bottom slope, $S_o = 0.0008$. Assume that the channel has a concrete lining with a trowel finish. From Table 4.2, $n = 0.012$, from Table 4.1.1 the area is

$$A = by + zy^2 = (10)(5.4) + 2.5(5.4)^2$$

$$A = 54 + 72.9 = 126.9 \text{ ft}^2$$

The wetted perimeter is

$$P = b + 2y\sqrt{1 + z^2} = 10 + 2(5.4)\sqrt{1 + 2.5^2} \quad P = 39.08 \text{ ft.}$$

$$R = \frac{A}{P_w} = \frac{126.9}{39.08} = 3.25 \text{ ft.}$$

Substituting in Equation 4.2.1 gives

$$Q = \frac{1.49}{(0.012)} (126.9) (3.25)^{2/3} (0.0008)^{1/2}$$

$$Q = 977.84 \text{ cfs}$$

4 2 1 - The Section Factor for Uniform Flow Computation

The expressions $A(R)^{2/3}$ is called the section factor for uniform flow computation. By rearranging the terms of Equation 4.2.1 the following expression is found.

$$SF = A(R)^{2/3} = \frac{n Q}{1.49 \sqrt{S}} \quad (4.2.2)$$

The left side of this equation depends only on the geometry of the section. Since $A(R)^{2/3}$ is a function of the normal depth y_n , the values of y_n versus $A(R)^{2/3}$ may be graphed for various channel geometries

The right-hand side of the equation depends upon the physical characteristics of the channel (n and S) and the flowrate (Q). Therefore, the equation shows that for a given geometric section there is only one possible depth for maintaining uniform flow, provided that the values of $A(R)^{2/3}$ always increase with depth, which is true in most cases (circular sections being the notable exceptions)

Equation 4 2 1 is a very useful tool for the computation and analysis of uniform flow. With a given discharge, slope and roughness, a section factor $A(R)^{2/3}$ and hence the normal depth y_n may be found.

Figure 4.2 1 has been provided to simplify the computations. This figure has dimensionless curves which show the relationship between the normal depth and section factor for rectangular, trapezoidal and circular sections. Example 4 2 1 illustrates the use of this figure.

Example 4.2.3

Solve Example 4.2 1 using the section factor method.

From Equation 4 2 2 (b = bottom width of channel)

$$A(R)^{2/3} = \frac{(225) (0.015)}{1.49(0.0007)^{1/2}} = 85.61; \text{ then}$$
$$\frac{AR^{2/3}}{b^{8/3}} = \frac{85.61}{10^{8/3}} = 0.18$$

From Figure 4.2 1 with $z = 2$, $y_n/b = 0.32$ and $y_n = 3.16$ ft. This is the same answer found in Example 4 1 1.

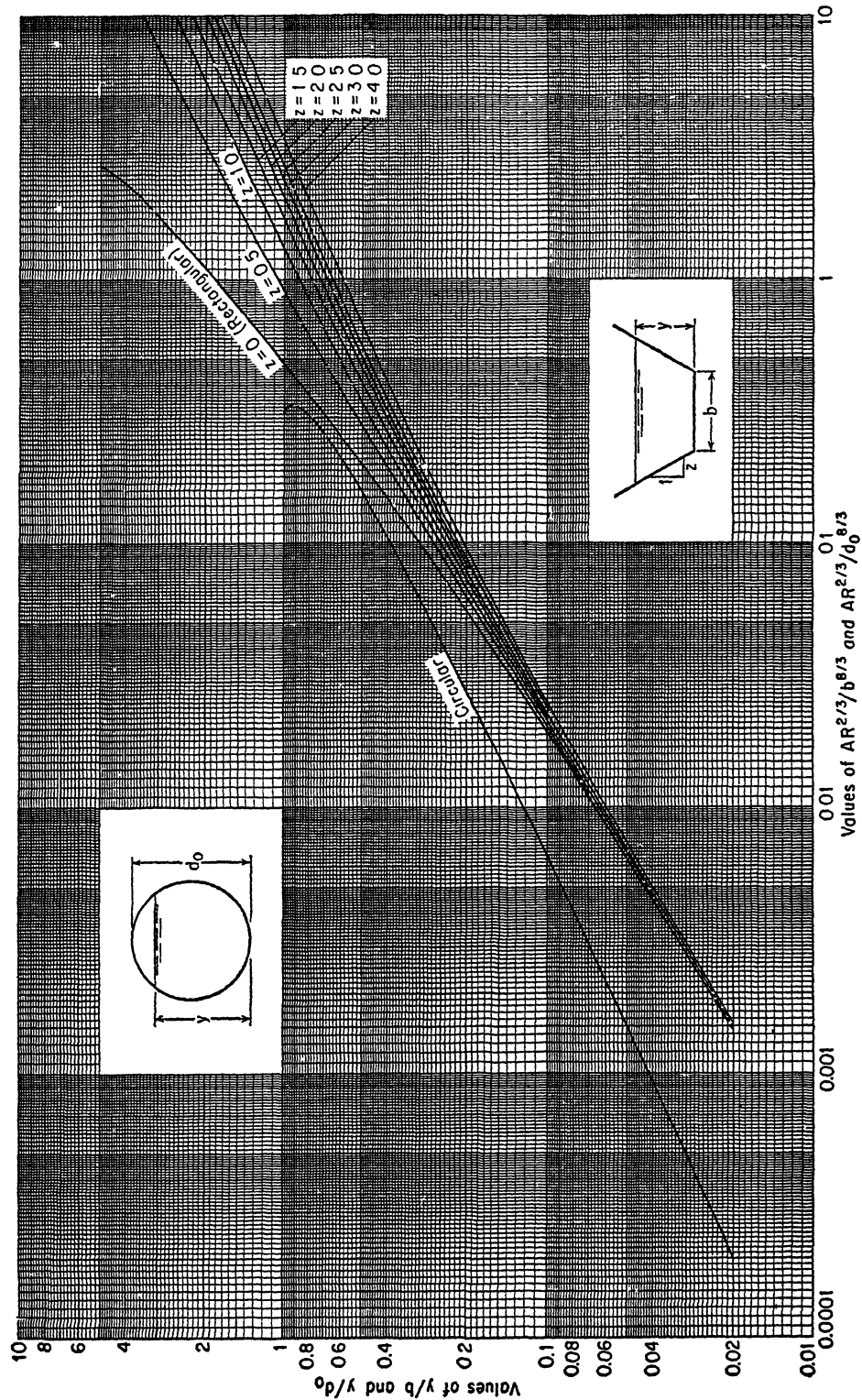


Figure 4.2.1
Normal Depth (y_n) for Uniform Flow in Open Channels (Chow, 1988)
[Note: $y = y_n$ in this graph]

4.2.2 - Specific Energy

Specific energy in a channel section is defined as the energy per weight of water flowing at any section of a channel, measured with respect to the channel bottom. Specific energy may be reduced to a dimension of feet. From Equation 4.1 2, it can be shown that the total energy at a section is equal to the sum of the potential and kinetic energy, if losses are neglected.

$$\frac{V^2}{2g} + y + z = \text{CONSTANT}$$

If the datum is taken at the channel bottom, the specific energy E is given by

$$E = y + \frac{V^2}{2g} \quad (4.2.3)$$

which indicates that the specific energy is equal to the sum of the velocity head and the water depth. From the continuity equation ($V = Q/A$) we can rewrite Equation 4 2 3 as:

$$E = y + \frac{Q^2}{2g(A)^2} \quad (4 2 4)$$

It is easily shown therefore, that the specific energy for a given channel and discharge is a function of the depth of flow only.

If the flow depth is plotted against the specific energy for a given channel section and discharge, a specific energy curve is obtained (Figure 4.2.2). This curve shows that for a given specific energy, there are two possible depths which correspond to that specific energy. It is also observed that at point C, a minimum specific energy occurs. It will be seen later that this point corresponds to a critical state of flow. At this point, the two alternate depths of flow apparently become one. This depth is termed the critical depth y_c .

When the depth of flow is larger than the critical depth, the velocity of flow is less than the critical velocity, and this flow is called subcritical. It should be noted that at or near the critical depth, the flow is unstable. This is because a minor change in the specific energy in this region will cause a major change in depth. Thus flow near the critical depth is often wavy.

If the discharge changes, the specific energy will change accordingly as shown in Figure 4 2 2. It will be seen later that as the channel slope is increased, for a given discharge and cross-sectional shape, the flow will change from subcritical, to critical and finally to supercritical.

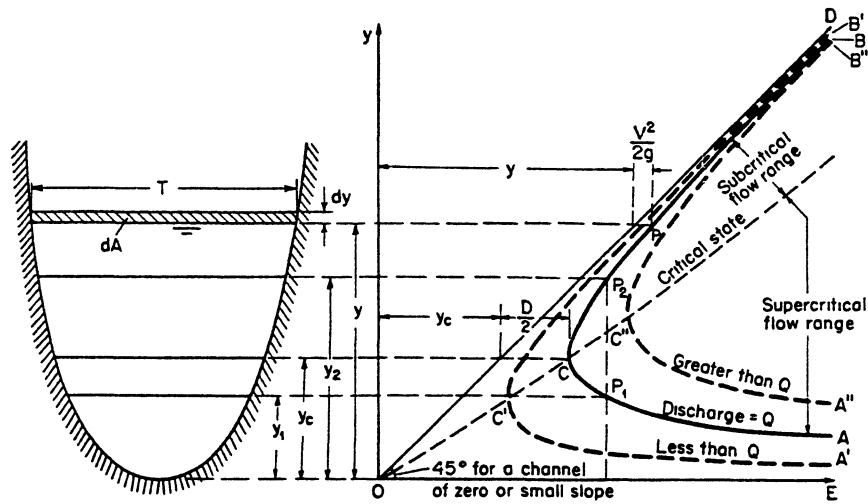


Figure 4.2.2
Specific-Energy Curve (Chow, 1988)

4.2.3 - Criterion for Critical State of Flow

The critical state of flow has been defined as the condition for which the Froude number is equal to 1 (Section 4.1.2). This occurs when the specific energy is a minimum for a given discharge. A theoretical criterion for critical flow may be developed as follows.

If Equation 4.2.4 is differentiated with respect to y , and noting that Q is a constant, Equation 4.2.5 is obtained

$$\frac{dE}{dy} = \frac{Q^2}{gA^3} \frac{dA}{dy} = 1 - \frac{V^2}{gA} \frac{dA}{dy} \quad (4.2.5)$$

The differential water area dA near the surface is equal to Tdy . We have seen that the hydraulic depth $D = A/T$ and since $dA/dy = T$, Equation 4.2.5 becomes

$$\frac{dE}{dy} = 1 - \frac{Q^2 T}{gA^3} = 1 - \frac{Q^2}{gA^2 D} = 1 - \frac{V^2}{gD} \quad (4.2.6)$$

At the critical state of flow, the specific energy is a minimum ($dE/dy = 0$).

$$\frac{V^2}{2g} = \frac{D}{2} \quad (4.2.7)$$

This is the criterion for critical flow, which simply states that at the critical flow state the velocity head is equal to one half the hydraulic depth. Equation 4.2.7 may also be written as $V^2/gD = 1$ (which as before states that the Froude number is equal to one)

Solving Equation 4.11 in a different manner for $dE/dy = 0$, the following expression is obtained

$$\frac{Q^2}{gA^2D} = 1 = \frac{Q^2T}{gA^3} \quad (4.2.8)$$

Solving for Q gives

$$Q = \left[\frac{g A^3}{T} \right]^{1/2} = [g A^2 D]^{1/2} \quad (4.2.9)$$

Equation 4.2.9 demonstrates that at critical conditions, the flow depends upon the flow depth only, and is not dependent upon the slope, roughness or water properties.

4.2.4 - Computation of Critical Flow Depth

Since the computation of critical depth will be of importance in later work, several methods for its computation will be investigated

The section factor SF for critical-flow computation was defined on page 4 - 2 as $SF = A\sqrt{D}$. If this is substituted into Equation 4.2.9, then the following expression is obtained

$$SF = \frac{Q}{\sqrt{g}} \quad (4.2.10)$$

Equation 4.2.10 is very useful in the computation of critical flow in open channels. For a given discharge, the section factor and consequently the depth may be determined. While on the other hand, if the depth is given, z and the critical discharge may be determined

Figure 4.2.3 is an aid in these computations. These are dimensionless curves of z versus the critical depth y_c , for circular, rectangular and trapezoidal shapes. Example 4.2.2 illustrates the computation of critical depth, with and without the use of Figure 4.2.3.

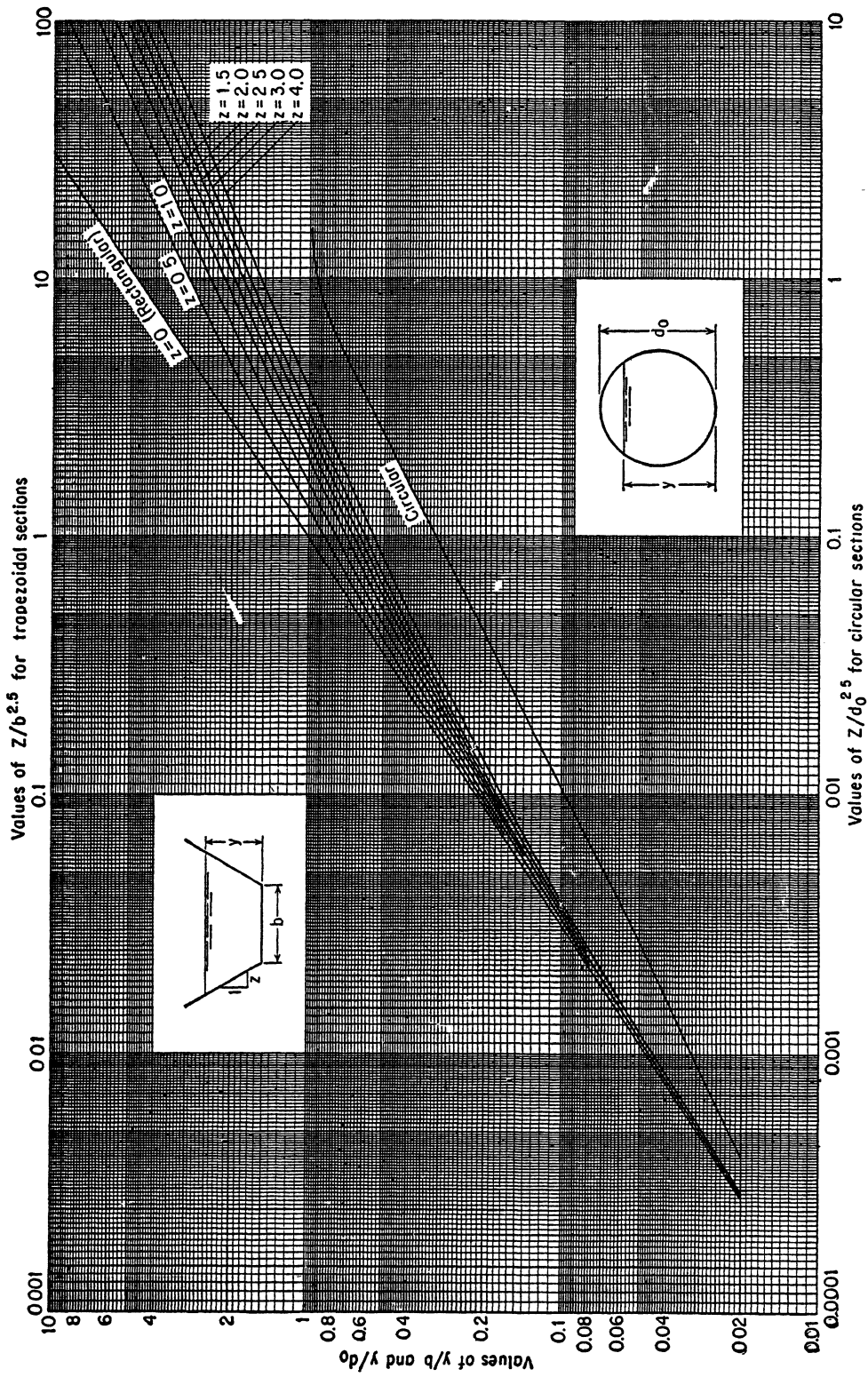


Figure 4.2.3

Curves for Determining the Critical Depth (y_c) in Open Channels (Chow, 1988)

- [Note:
- (a) $y = y_c$ in this graph
 - (b) Use the bottom scale of graph for circular sections and top scale for trapezoidal sections]

Example 4.2.4

For the channel given in Example 4.1.1, calculate the critical depth for a flow of 225 cfs and $n = 0.015$ by trial and error, and by using Figure 4.2.3

As before.

$$A = (10 + 2y)y$$

$$T = 10 + 4y$$

$$P_w = 10 + 2y\sqrt{5}$$

$$D = \frac{A}{T} = \frac{(10 + 2y)y}{(10 + 4y)}$$

$$R = A/P_w$$

From Equation 4.2.9

$$Q = 225 = [gA^2D]^{1/2} = \frac{32.2^{1/2}[(10 + 2y)y]^{3/2}}{(10 + 4y)^{1/2}}$$

Solving by trial and error a critical depth, y_c , is found to be 2.16 ft

This problem may also be solved using Figure 4.2.3. The section factor, SF, is:

$$SF = \frac{Q}{\sqrt{g}} = \frac{225}{\sqrt{32.2}} = 39.65$$

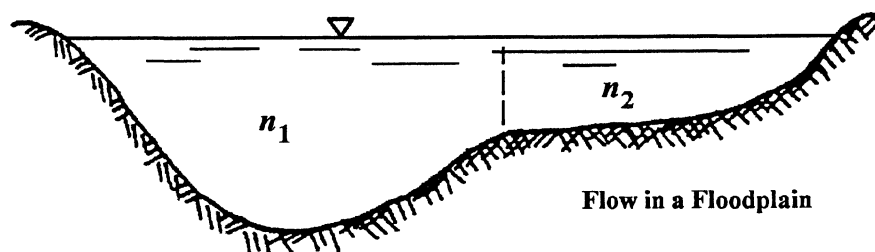
and

$$\frac{SF}{b^{2.5}} = \frac{39.65}{10^{2.5}} = 0.125$$

From Figure 4.2.3 with $z = 2$, $y_c/b = 0.23$, thus $y_c = 10(0.23) = 2.30$ ft.

4 2 5 - Steady Uniform Flow in a Floodplain

A practical open channel problem of importance is the computation of discharge through a natural floodplain. The figure below shows a hypothetical case.



In general, the floodplain is rougher than the river channel, and the depth is much less. The slopes of the energy grade-line for the two are the same and the discharge is determined for each section separately. Using the dashed line in the above figure, the discharges for each section are computed separately to determine the total discharge of the system. Since both sections have the same slope, the total discharge may be expressed as:

$$\begin{aligned} Q &= Q_1 + Q_2 \\ &= K'_1 \sqrt{S} + K'_2 \sqrt{S} \\ &= (K'_1 + K'_2) \sqrt{S} \end{aligned} \quad (4.2.11)$$

$$\text{where } K'_i = \frac{1.49}{n_i} A_i R_i^{2/3} \quad (4.2.12)$$

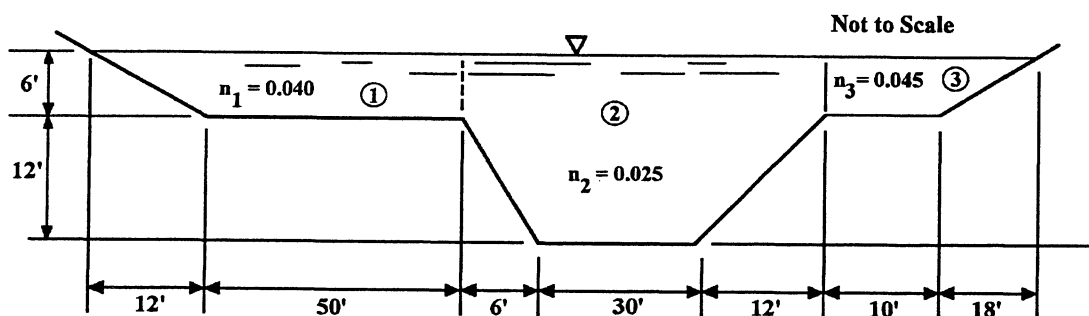
For a particular section shape, K' is a function of Mannings' n and the depth only. Example 4 2 3 illustrates the use of these equations.

The floodway is defined as the channel of a stream plus any adjacent floodplain areas, that must be kept free of encroachment so that the 100-year flood can be carried without substantial increases in flood heights. Minimum Federal standards limit such increases to 1.0 foot, provided that hazardous velocities are not produced (FEMA, 1984). In Indiana, however, state regulation limits such increases in flood heights to 0.14 feet. The portion of the floodplain outside of the floodway but still subject to flooding is referred to as "floodway fringe."

Example 4.2.5

Determine the total discharge for the floodplain shown below. The channel slope is 0.0006 ft/ft. Compute the value of K' for each section.

$$K'_i = \frac{1.49}{n_i} A_i R_i^{2/3}$$



* Section 1

$$A_1 = 50 \times 6 + 1/2(12 \times 6) = 336 \text{ ft}^2$$

$$P_{w1} = 13.42 \text{ ft} + 50 \text{ ft} = 63.42 \text{ ft}$$

$$R_1 = \frac{A_1}{P_{w1}} = \frac{336 \text{ ft}^2}{63.42 \text{ ft}} = 5.30 \text{ ft}$$

$$K'_1 = \frac{1.49}{0.040} (336)(5.30)^{2/3} = 38,046.62$$

* Section 2

$$A_2 = 6(30 + 6 + 12) + 12(30) + 1/2(12)(12) + 1/2(6)(12) = 756 \text{ ft}^2$$

$$P_{w2} = 16.97 + 30 + 13.42 = 60.39 \text{ ft}$$

$$R_2 = \frac{A_2}{P_{w2}} = \frac{756 \text{ ft}^2}{60.39 \text{ ft}} = 12.52 \text{ ft}$$

$$K'_2 = \frac{1.49}{0.025} (756) (12.52)^{2/3} = 242,942.94$$

* Section 3

$$A_3 = (10)(6) + 1/2(18)(6) = 114 \text{ ft}^2$$

$$P_{w3} = 10 + 18.97 = 28.97 \text{ ft}$$

$$R_3 = \frac{A_3}{P_{w3}} = \frac{114}{28.97} = 3.94 \text{ ft}$$

$$K'_3 = \frac{1.49}{0.045} (114)(3.94)^{2/3} = 9,416.21$$

The total flow therefore is equal to

$$Q = (K'_1 K'_2 + K'_3) \sqrt{S} = (38046.58 + 242944.8 + 9416.21) \sqrt{0.0006} = 7,113.50 \text{ cfs}$$

4.3 - DESIGN CRITERIA FOR UNIFORM FLOW

4.3.1 - Site Selection

Drainage channels provide surface drainage and must be placed where they will adequately perform their drainage functions. In rural areas, as well as urban areas, topography, location of highways, structures, and other obstacles largely fix the location, alignment, and grade of the channel and determine the quantity of surface water entering it.

Design of the channel is dependent upon two principles. The first principle dictates that the design be in harmony with the physical laws and properties that control such items as topography, construction materials, and the hydraulics itself. Although the first two items are rather obvious, the last one has hidden implications. Although a cross-section can be determined to handle flows for normal conditions, more than often, these same cross-sections will be used for conditions that do not strictly apply. Designing a channel with a sharp curve using the exact same design section as the straight-run portion could lead to erosion of the curve to the point of failure. Assuming the flow as being uniform (in fact it is gradually varied) could in some instances result in the channel being overtopped somewhere upstream.

The second principle points out that the design layout of the drainage system requires knowledge and experience relating to both construction and maintenance. Construction involves the initial capital investment relating suitable channel sections, necessary safety precautions, types of channel lining for erosion protection, and aesthetics. Maintenance assures that the desired function of the channel survives. Consideration of both costs and benefits must be evaluated before any final decision may be concluded.

The actual layout of the drainage system should preferably be made on a topographic map which contains the location of all obstructions, and the accentuated ridge and drainage lines. In most cases it is sufficient to follow the natural grade lines of the drainage paths while in some instances these natural drainages must be intercepted and altered.

The approximate grade of the channel is computed from the topographic map. The grade affects both the size of the channel required to carry a given flow and the velocity at which the flow occurs. The flow should be subcritical whenever possible. Changes in channel alignment should be as gradual as the width of right-of-way and terrain permit. Whenever practicable, changes in alignment should be made in the reaches of the channel which have flatter slopes, particularly if the flow becomes supercritical on the steeper slopes.

Selection of the type of channel lining used; grass, earth, or artificial, can influence the location. An impermeable lined channel could be constructed in a sandy area whereas another type could not. The eventual selection must be based on the function or purpose of the channel itself. This choice must consider alternate proposals that achieve the same goals.

4.3.2 - Channel Cross-section Selection

For a channel conveying a given discharge for a given slope and Manning's roughness coefficient, some cross-section shapes are more efficient than others. When a channel is constructed, the costs are a function of the excavation and lining, both of which should be minimized. Based on the Manning equation, it can be shown that when the area of cross-section is minimum, the wetted perimeter is also minimum. This implies that both lining and excavation costs approach their minimum value for the same dimensional channel. Therefore, the best hydraulic section for a lined channel is one that has the least wetted perimeter, or its equivalent, and the least area for the type of section.

The geometric elements of six best hydraulic sections are listed below in Table 4.3.1. These sections may not always be practical due to the difficulties in construction and in the use of materials. In general, a channel selection by use of the best hydraulic efficiency gives a good starting point for design. Modifications for practicability, safety, or other imposed limits could be applied to these cross-sections. Because Table 4.3.1 is not in a directly usable form, and due to the fact that most channels are often designed in the shape of trapezoids, a nomograph method of design is presented in this section under "Design Procedures" (Section 4.3.5) that will increase the efficiency of the designer. The nomograph procedure is also helpful because

the "best" section can be determined at various side-slopes that deviate from the optimum.

Table 4.3.1
Best Hydraulic Sections (Chow, 1988)

Cross section	Area A	Wetted perimeter P	Hydraulic radius R	Top width T	Hydrau- lic depth D	Section factor Z
Trapezoid, half of a hexagon	$\sqrt{3} y^2$	$2 \sqrt{3} y$	$\frac{1}{2} y$	$\frac{4}{3} \sqrt{3} y$	$\frac{3}{4} y$	$\frac{3}{2} y^2 \text{ }^5$
Rectangle, half of a square	$2y^2$	$4y$	$\frac{1}{2} y$	$2y$	y	$2y^2 \text{ }^5$
Triangle, half of a square	y^2	$2 \sqrt{2} y$	$\frac{1}{4} \sqrt{2} y$	$2y$	$\frac{1}{2} y$	$\frac{\sqrt{2}}{2} y^2 \text{ }^5$
Semicircle	$\frac{\pi}{2} y^2$	πy	$\frac{1}{2} y$	$2y$	$\frac{\pi}{4} y$	$\frac{\pi}{4} y^2 \text{ }^5$
Parabola, $T = 2 \sqrt{2} y$	$\frac{4}{3} \sqrt{2} y^2$	$\frac{8}{3} \sqrt{2} y$	$\frac{1}{2} y$	$2 \sqrt{2} y$	$\frac{2}{3} y$	$\frac{8}{9} \sqrt{3} y^2 \text{ }^5$
Hydrostatic catenary	$1.39586y^2$	$2.9836y$	$0.46784y$	$1.917532y$	$0.72795y$	$1.19093y^2 \text{ }^5$

One other factor might affect final cross-section selection. The best hydraulic section gives the minimum excavation only if the water surface is at the level of the bank top. Where the water surface is below the bank top, channels narrower than those of the best hydraulic section will result in minimum excavation.

Figure 4.3.1 gives freeboard heights that will take into account factors such as wave action due to wind that create the need for such additional depths of channels. Figure 4.3.1 also provides recommendations on the height of any artificial lining that may be used. Freeboard is recommended for all channel designs.

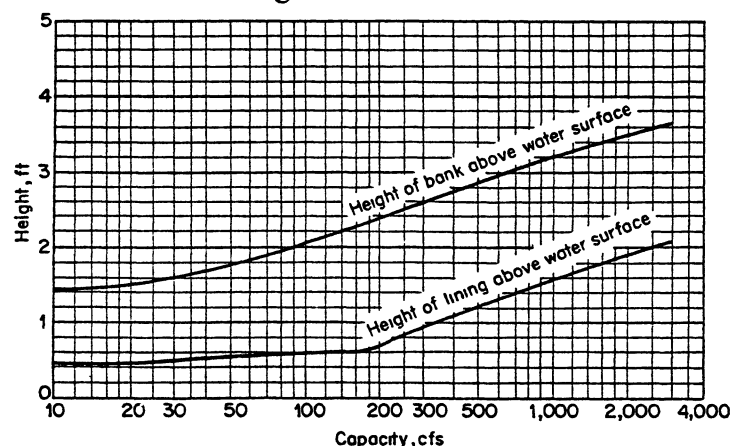


Figure 4.3.1
Recommended Freeboard and Height of Bank for Lined Channels
(U S Bureau of Reclamation, 1952)

Up to this point, the principle of best hydraulic section has been applied only to the design of non-erodible lined channels. In channels of this nature, high velocities may be tolerated. For erodible channels, the best hydraulic section must be modified to take into account the limits placed on excessive velocities.

In addition to the above criteria relating to channel cross-section selection, other factors must be considered. In urban areas, safety consideration may dictate the maximum side slopes and depths of flow which are desirable. If the channel is grassed and mowing of this side slope is necessary, the maximum slope is dictated by the ease with which it can be maintained.

4.3.3 - Roughness Coefficients

The greatest difficulty in using any of the various formulas for calculating the discharge capacity of channel, is the determination of a roughness coefficient. Since this manual makes extensive use of the Manning equation and because Manning's roughness coefficient n may be related to all other coefficients, the following discussion applies to selecting an appropriate value for Manning's n from Table 4.2.1.

A selection of a value of n implies that one is attempting to estimate the resistance to flow in a given channel. Before one can make any decision on that selection, the various factors that affect resistance must be understood. The first factor must be surface roughness. In one sense, surface roughness is represented by the size and shape of the grains of the material forming the wetted perimeter and producing a retarding effect on the flow. In a lined channel it is based on the artificial surface; whereas in a natural channel, it is based on the composition of grain sizes that make up the bed and sides.

Vegetation may also be regarded as a kind of surface roughness since it also produces a retarding effect on the flow. This effect is a function of the height, density, distribution, and type of vegetation. Any natural or artificial phenomena that affects the pattern or state of growth would also affect the roughness coefficients. The most prominent would be seasonal change. In this instance, n may increase in the growing season and diminish in the dormant season.

Obstructions in the channel are also regarded as a surface roughness, but not directly linked to the channel itself. Such debris may be removed, but in any case the effect is identical: retardation of the flow. The amount of increase in n , and therefore decrease in flow, depends on the nature of the obstructions, their size, shape, number, and distribution. A side effect of obstructions, other than surface roughness, is the creation of vortices and eddies that would cause an increase in turbulence and scour, and therefore, a decrease in available energy for transportation of flow. Such conditions would appear as an increase in the value of n .

Moving suspended material and bed load requires energy above that necessary to transport the water itself. The direct effect of the consumption of energy is to cause an additional head loss and, therefore, cause an apparent increase in channel roughness. Even when the bed load is not moving, the increase in obstructions causes increased turbulence. In either case, one would expect an increase in the value of n .

Up to this point, the channel alignment was considered straight. Smooth curvature with large radii will provide a relatively low increase of n , whereas sharp curvature with severe meandering will provide high increases of n . The degree of meandering of a channel course dictates the final correction along with the type of material used. For artificial channels it has been suggested that the value of n be increased 0.001 for each 20 degrees of curvature in 100 feet of channel. The meandering of natural streams, on the other hand, may cause extensive variation of n . For severe meandering the n value is usually increased by 30 percent, whereas for appreciable meandering the increase is 15 percent.

For natural channels, the coefficient of roughness is a function of stage and discharge. When the stage and discharge are low, the irregularities of the channel are exposed and their effects pronounced. As the stage increases, the sides or banks could relatively increase or decrease the roughness, depending on the composition and presence of obstructions. If water stage becomes too high, the water will flow out of its banks and thus change the relative roughness again. As described in an earlier section, it is best to break the channel into separate sections and simply find a mean for the value of n .

4 3 4 - Design Practices

Open channels may be broadly classified into rigid, lined, non-erodible channels and erodible channels which may be lined with grass, rip-rap, or even the bare soil itself. The design procedures for these two classes are distinctly different. Non-erodible, lined channels are the easiest to design. The major factors to consider are the kind of material forming the channel lining, the minimum velocity to avoid deposition if the water carries silt or debris, the channel bottom slope and side slopes, the freeboard, and the most efficient section.

The purpose of the lining is usually to prevent erosion, but in some instances, it may be employed to check seepage losses. Normally, excessive velocities can be ignored unless the water carries sand or gravel that may erode the strongest of channel linings, or unless the velocity is so high that there is a tendency of the water to pick up and move lining material. Such high velocities are extremely dangerous from the standpoint of safety also.

The minimum velocity is the lowest velocity that will prevent either the start of sedimentation or the growth of aquatic plants and moss. This velocity is very uncertain and exact values are dependent upon many factors. What has generally been accepted, is to maintain a minimum mean velocity of three feet per second when the percentage of silt is small.

Channel bottoms and side slopes are usually restricted by the amount of space that is available. In populated areas, flat side slopes are safer on two counts. If someone should fall into the channel it is easier for that person to get out if the side slopes are flat. Also with flat side slopes, any vehicle that should stray into the channel may exit with minimal damage to the driver or the channel. The final selection of the side slopes and bottom width will dictate the depth of flow in the channel. This may be important as far as aesthetics or other intangible design inputs.

The freeboard of a channel is the additional vertical distance from the design water surface to the top of the channel. Besides providing additional capacity, its major role is to provide sufficient height to prevent waves or fluctuation in water surface from overflowing the sides. While no universally accepted rule exists for the determination of freeboard, the curves presented in Figure 4.3.1 represent a reasonable approach.

Figure 4.3.1 also provides a guide to the height of lining that should extend above the water surface. If the underlying soil is susceptible to erosion, then the channel lining or some other stabilizing material must continue to the top of the channel.

The design of erodible channels presents the engineer with the greatest challenge. Most channels, for cost reasons, must be unlined and it is necessary therefore that they be so proportioned as to prevent silting or scouring in objectionable quantities. The velocities of flow at all points of the cross-section must be sufficient to transport through the channel all sediment that enters it, however, the velocities must be low enough, at all points, to prevent scouring of erodible linings.

The present state of sedimentation theory is inadequate to permit a precise design for channels, but it is advanced far enough to give one a workable methodology. Channel design however, must still be based in large measure on engineering experience and judgement.

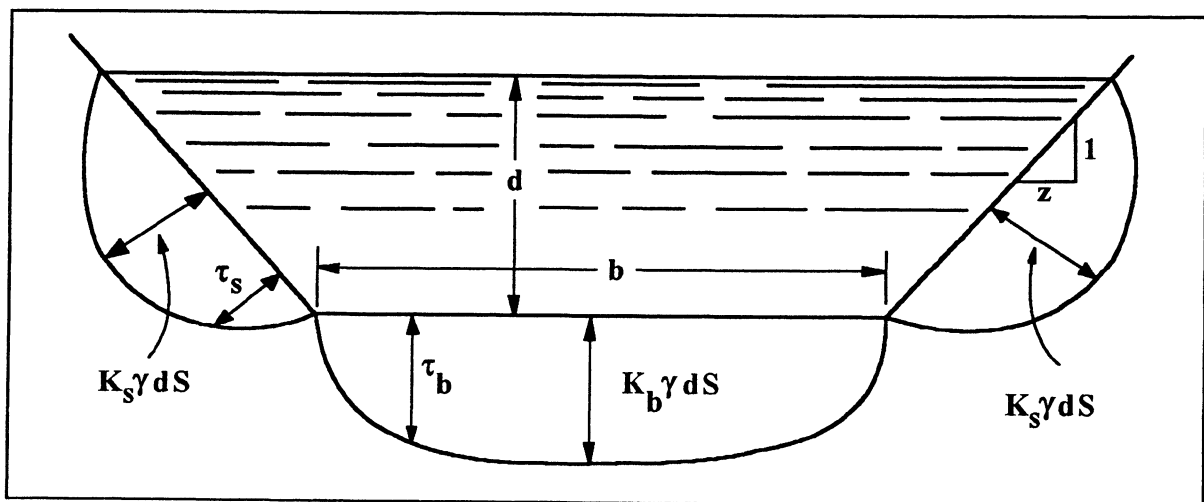


Figure 4.3.2
Tractive Stress Distribution on Channel Bed

A method based on the concept of tractive force has been developed by engineers and consultants of the U.S. Bureau of Reclamation. The rationale of the solution is simple. When water flows in a channel, a force is exerted that acts in the direction of flow on the channel bed. This force, which is simply the pull of water on the wetted surface, is known as the tractive force.

Tractive force can be shown to equal

$$\tau_o = \gamma RS \quad (4.3.2)$$

where τ_o = the average bed shear force, tractive force (lb/ft²)

This is the average unit tractive force whereas the magnitude actually varies along the wetted perimeter as shown in Figure 4.3.2.

The coefficients K_s and K_b , for maximum shear on the sides and bottom, respectively, depend on the width-depth ratio, b/d , and the side-slope z . However, for channels of ordinary size and shape (where b/d is greater than 3.0) the values of K_s and K_b may be taken as 0.75 and 1.0, respectively.

Thus, the maximum tractive force on the side-slope is only about 3/4 that on the bottom. This might imply that only the bottom is the critical point and one can ignore sides. However, motion of soil particles on the side-slope is also assisted by gravity, so that the resultant effect of tractive force and gravitational force is often such as to cause the critical point for incipient scour to be on the side rather than the bottom.

It can be shown that the ratio of the tractive force on the sides to that on the bottom is equal to

$$K = \tau_s / \tau_b \quad (4.3.3)$$

$$K = \left[1 - \frac{\sin^2 \phi}{\sin^2 \theta} \right]^{1/2} \quad (4.3.4)$$

where ϕ = angle of the side-slope and θ = angle of repose for the soil.

Therefore, by calculating the permissible tractive force on the bottom, it is easy to determine the permissible tractive force on the sides. Equating this with the maximum tractive force on the channel section, a workable design can be developed.

A modification to the tractive force approach for erodible channel design is the concept of maximum permissible depth of flow. An examination of Equation 4.3.2 indicates that the

maximum tractive force that can exist without erosion occurring for a particular lining material depends directly on the hydraulic radius. Since the hydraulic radius and depth are related, it is possible to define a maximum permissible depth of flow which can occur before erosion takes place. This approach, which is a more convenient design procedure, will be used in this manual

For wide channels of any shape (and for a given channel slope, depth, and lining), the vertical velocity distribution in the central and deepest section, where wall effects are negligible, should be identical. Also, the first scour occurs at the deepest portion of the channel since the wall or bottom shear stress is greatest in that portion.

If the depth of flow, channel slope, lining, and soil are the same in both channels shown in Figure 4.3.3, then the flow rate and the mean channel velocity for the two channels will be different. But in the central section of both channels, represented by ΔX , the velocity distribution and bottom shear stress will be nearly identical. Therefore, in both channels, there exists a limiting depth of flow above which scour will occur, and this depth, d_{max} , is the same for all wide channels of the same longitudinal slope, lining, and underlying soil. This concept has been verified by tests at Mississippi State University (1968), the Louisiana Department of Highways (1971) and by Anderson (1973). Thus, for unlined channels and rip-rapped channels, charts can be prepared which display the maximum depth of flow as a function of the channel slope and the degree of erodibility of the soil or the size of the rip-rap used. Such charts, which are taken from Normann (1975) are shown in Figures 4.3.4 and 4.3.5. The use of these charts are illustrated by Examples 4.3.1 and 4.3.2.

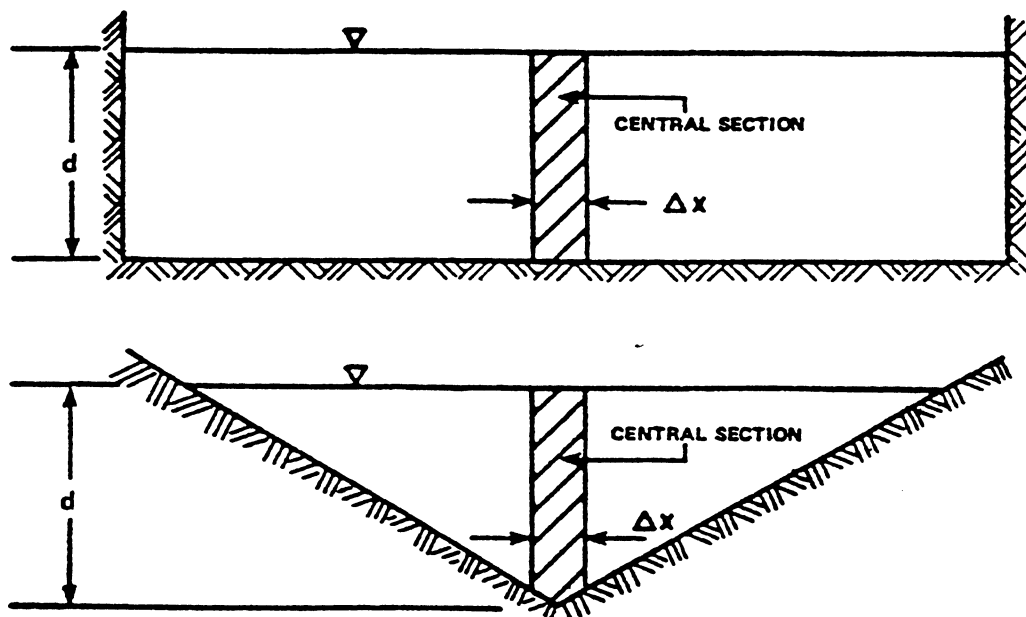


Figure 4.3.3
Schematic Diagram of Channels of Different Shapes (Normann, 1975)

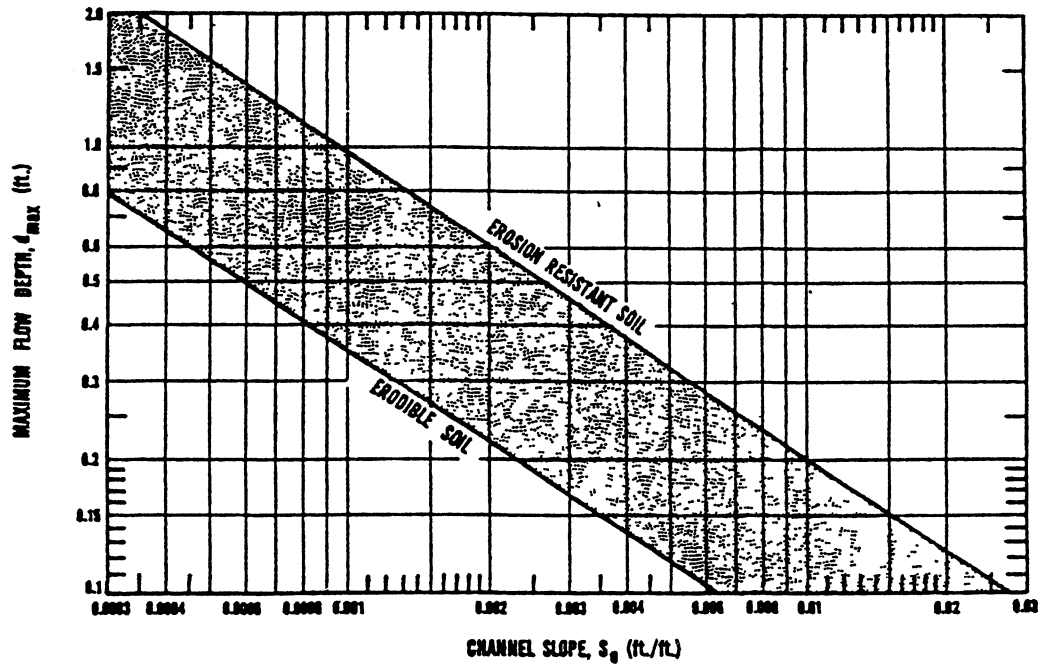


Figure 4.3.4
Maximum Permissible Depth of Flow (d_{max}) for Unlined Channels (Normann, 1975)

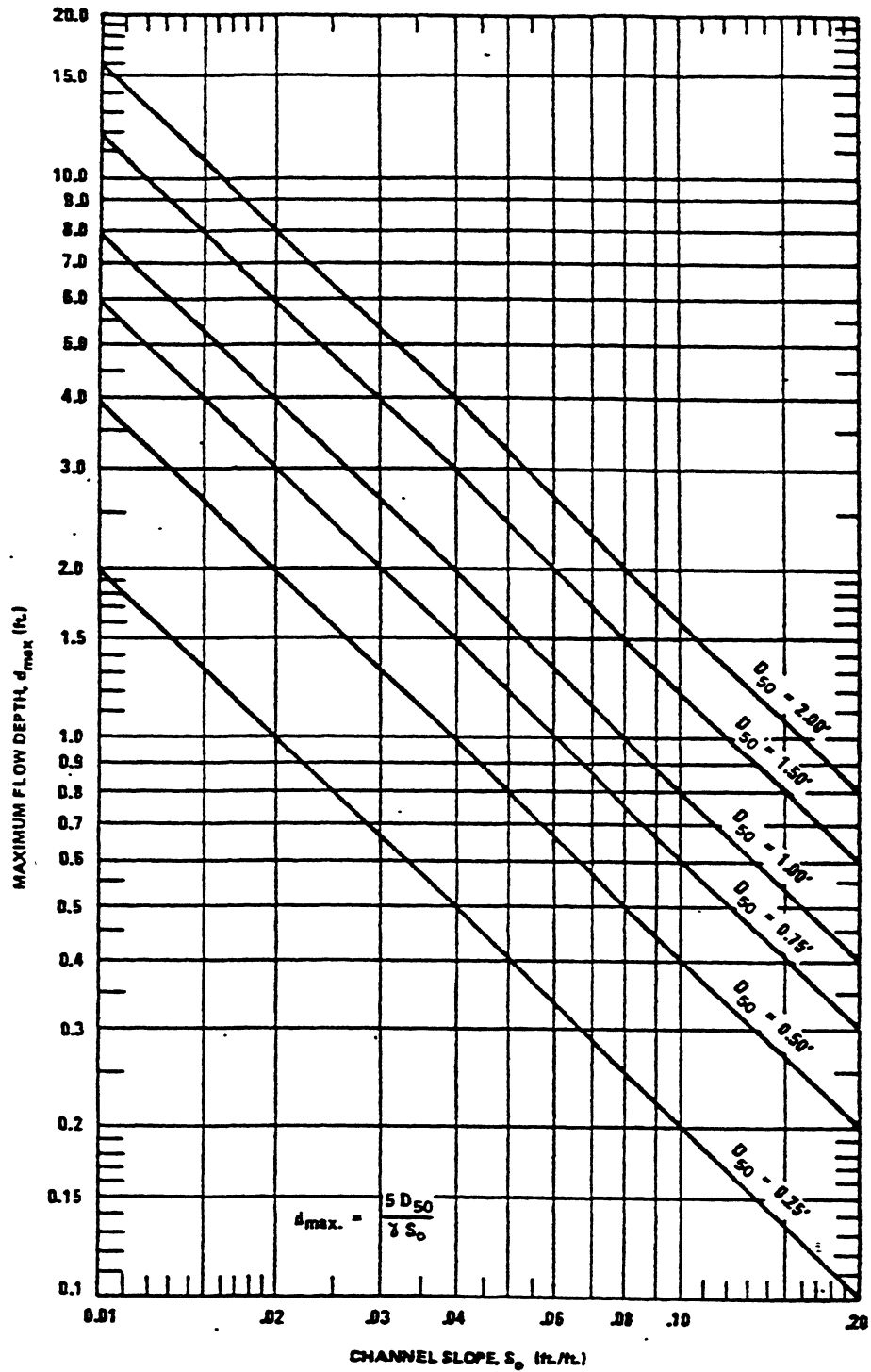


Figure 4 3.5
Maximum Permissible Depth of Flow (d_{max}) for
Channels lined with Rock Rip-rap (Normann, 1975)
Note: D_{50} is the particle size of gradation, of which 50% of the mixture is finer by weight.

The design of a grass-lined channel proceeds in somewhat the same manner. The resistance to flow in a grass-lined channel depends both upon the type of grass and its length. Work by the Soil Conservation Service (1954) has resulted in a classification scheme for grassed channels known as the degree of retardance. These groupings for various grasses, ranging from type A with very high retardance through type E with very low retardance, are shown in Table 4.3.2.

Table 4.3.2
Classification of Degree of Retardance for Various Kinds of Grasses (Chow, 1988)

Retardance	Cover	Condition
A Very high	Weeping love grass .	Excellent stand, tall (av 30 in.)
	Yellow bluestem ischaemum .	Excellent stand, tall (av 36 in.)
B High	Kudzu .	Very dense growth, uncut
	Bermuda grass	Good stand, tall (av 12 in.)
	Native grass mixture (little blue-stem, blue grama, and other long and short Midwest grasses)	Good stand, unmowed
	Weeping love grass .	Good stand, tall (av 24 in.)
	Lespedeza sericea	Good stand, not woody, tall (av 19 in.)
	Alfalfa	Good stand, uncut (av 11 in.)
	Weeping love grass	Good stand, mowed (av 13 in.)
	Kudzu	Dense growth, uncut
	Blue grama	Good stand, uncut (av 13 in.)
C Moderate	Crab grass	Fair stand, uncut (10 to 48 in.)
	Bermuda grass	Good stand, mowed (av 6 in.)
	Common lespedeza.	Good stand, uncut (av 11 in.)
	Grass-legume mixture—summer (orchard grass, redtop, Italian rye grass, and common lespedeza)	Good stand, uncut (6 to 8 in.)
	Centipede grass	Very dense cover (av 6 in.)
	Kentucky bluegrass .	Good stand, headed (6 to 12 in.)
D Low	Bermuda grass	Good stand, cut to 2.5 in. height
	Common lespedeza	Excellent stand, uncut (av 4.5 in.)
	Buffalo grass	Good stand, uncut (3 to 6 in.)
	Grass-legume mixture—fall, spring (orchard grass, redtop, Italian rye grass, and common lespedeza)	Good stand, uncut (4 to 5 in.)
	Lespedeza sericea	After cutting to 2 in. height, very good stand before cutting
E Very low	Bermuda grass	Good stand, cut to 1.5 in. height
	Bermuda grass .	Burned stubble

* U.S. Soil Conservation Service [41].

The ideal grass would be one that grows rapidly, but not over one inch in height, and develops a strong, deep-root system quickly. The selection of grass for a channel lining depends on the climate and the soil in which the plant is to survive. The development of the root system, which is dependent upon the individual plant, will add stability to the channel. In some cases it will be necessary to introduce two different types of grasses. One might be for fast establishment until a more permanent grass might grow out. It is most important to consider the condition of the grass after development. If the channel will be frequently cut, then a lower retardance or smaller roughness coefficient could be used in the Manning equation. This implies more capacity for the same cross-section.

Once the type of grass is selected, curves such as shown in Figure 4.3.7 (Normann, 1975) can be used to establish the maximum permissible depth of flow for some retardance classifications.

Again, according to the investigation by the Soil Conservation Service, it was found that the mean velocity of flow was related to the hydraulic radius R and the channel slope S_o . The curves which are presented in Figures 4.3.8 - 4.3.12 illustrate this relationship for the 5 types of vegetation retardance.

Example 4.3.1

Design a channel lining for a trapezoidal channel with a 4 foot bottom width and 4:1 side slopes. Design the permanent lining for a 10-year recurrence interval runoff. The soil has average erodibility. The channel top width is restricted to 12 feet. Channel slope is 1.5 percent. Prepare design analysis for several different permanent linings (bare soil, redtop grass, 3" rip-rap and concrete).

Detailed calculations are shown in Figure 4.3.6.

Note that the bare soil would convey very little on the 1.5 percent slope. Redtop grass 6 inches long is the best lining investigated.

DRAINAGE CHANNEL LINING DESIGN

PROJECT: EXAMPLE 4.3.1 **DATE:** 8/28/80
STATION: _____ **TO STATION:** _____ **DESIGNER:** CBB
DRAINAGE AREA = 25 **ACRES** **CHECKED BY:** _____
HYDROLOGIC COMPUTATIONS: **DATE:** _____

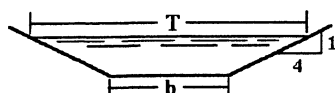
Given: $Q = 18$ cfs (for a 10-year return period)

DESIGN FLOW: Q 10 = 18 cfs

DESIGN FLOW FOR TEMPORARY LINING: Q --- = --- cfs

SOIL ERODIBILITY: Average

CHANNEL DESCRIPTION: MAX. TOP WIDTH = 12 ft



$$T = b + (z_1 + z_2) * d = 4 + 8d$$

$S_o =$ 0.015

AVAILABLE LININGS:

1. Bare Soil
2. Redtop Grass (4")
3. Riprap (3")
4. Concrete

	LINING	d_{max}	b	A	R	V	$Q = AV$	T	REMARKS
1.	Bare Soil	0.12	4	0.54	0.107	1.83	0.99	4.96	no good
2.	Redtop Grass	0.90	4	6.84	0.60	2.70	18.47	11.2	OK
3.	Riprap (3")	1.3	4	11.96	0.81	5.11	60.48	14.4	no good
4.	Concrete	1.0*	4	8	0.65	10.53	84.25	12	over designed high velocity
		* No d_{max}							

Figure 4.3.6
Sample Calculations for Example 4 3.1 (Normann, 1975)

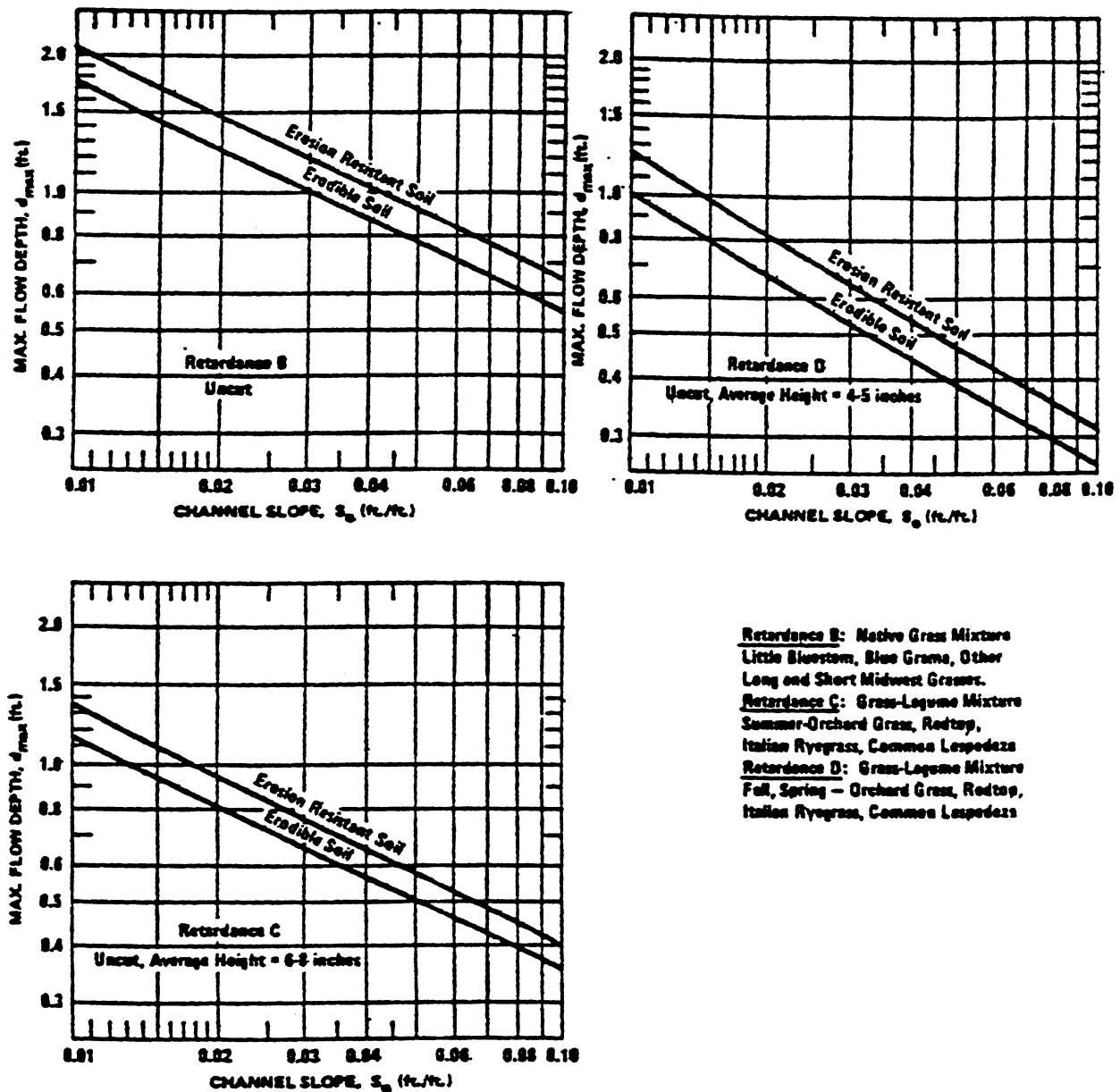


Figure 4.3.7
Maximum Permissible Depth of Flow (d_{max}) for Channels lined
with Grass Mixtures, Good Standing, Uncut (Normann, 1975)

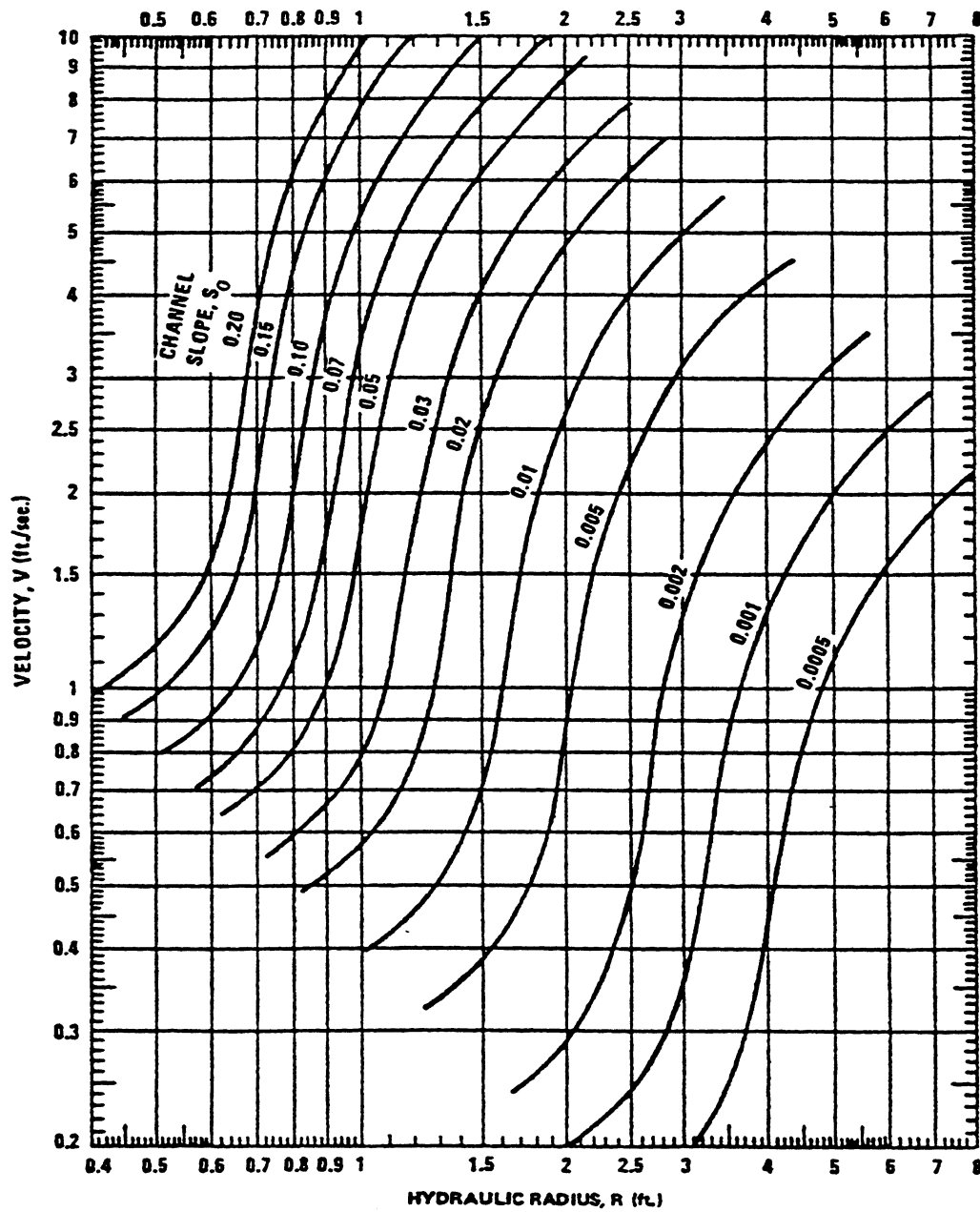


Figure 4 3.8
Flow Velocity for Channels Lined with Vegetation of Retardance A (Normann, 1975)

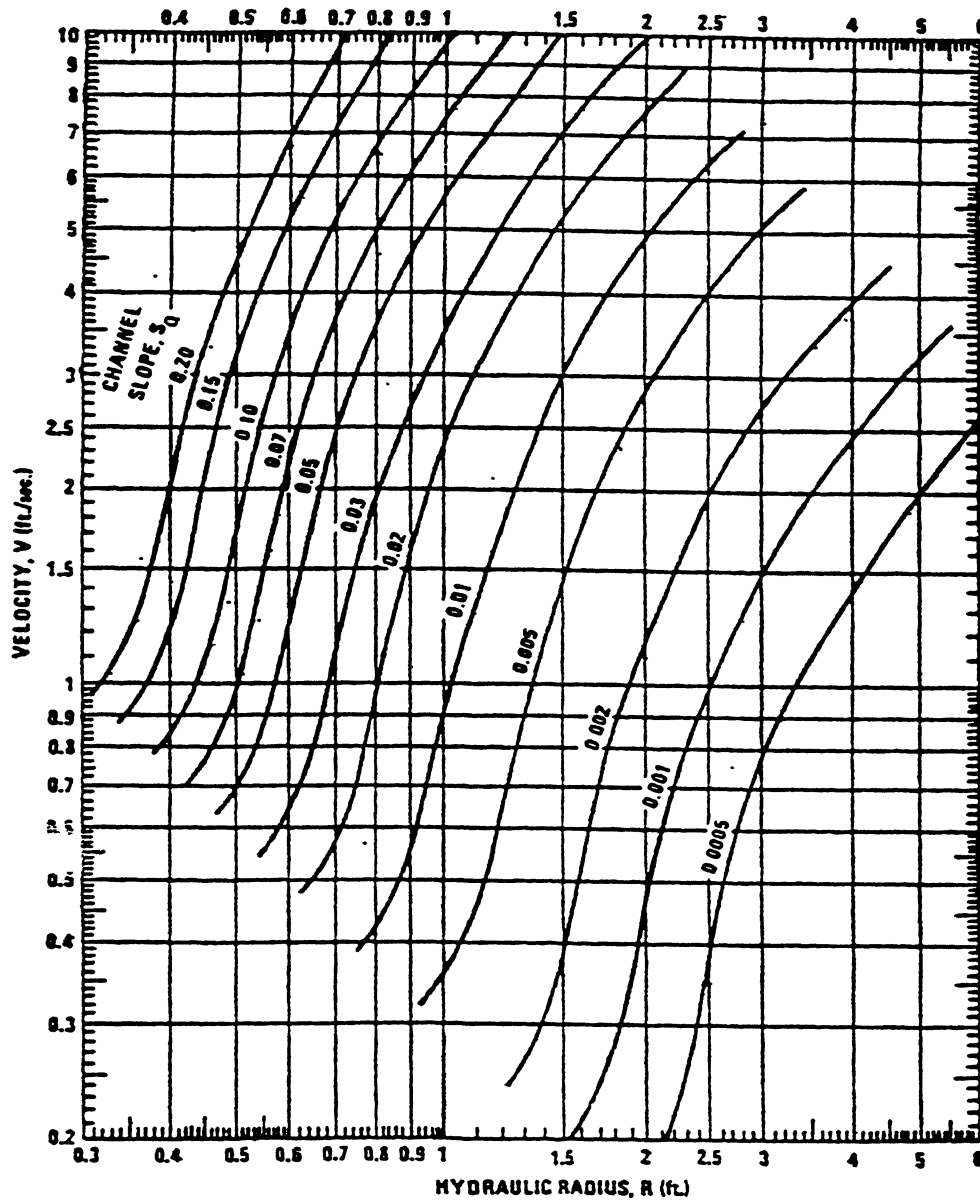


Figure 4.3.9
Flow Velocity for Channels Lined with Vegetation of Retardance B (Normann, 1975)

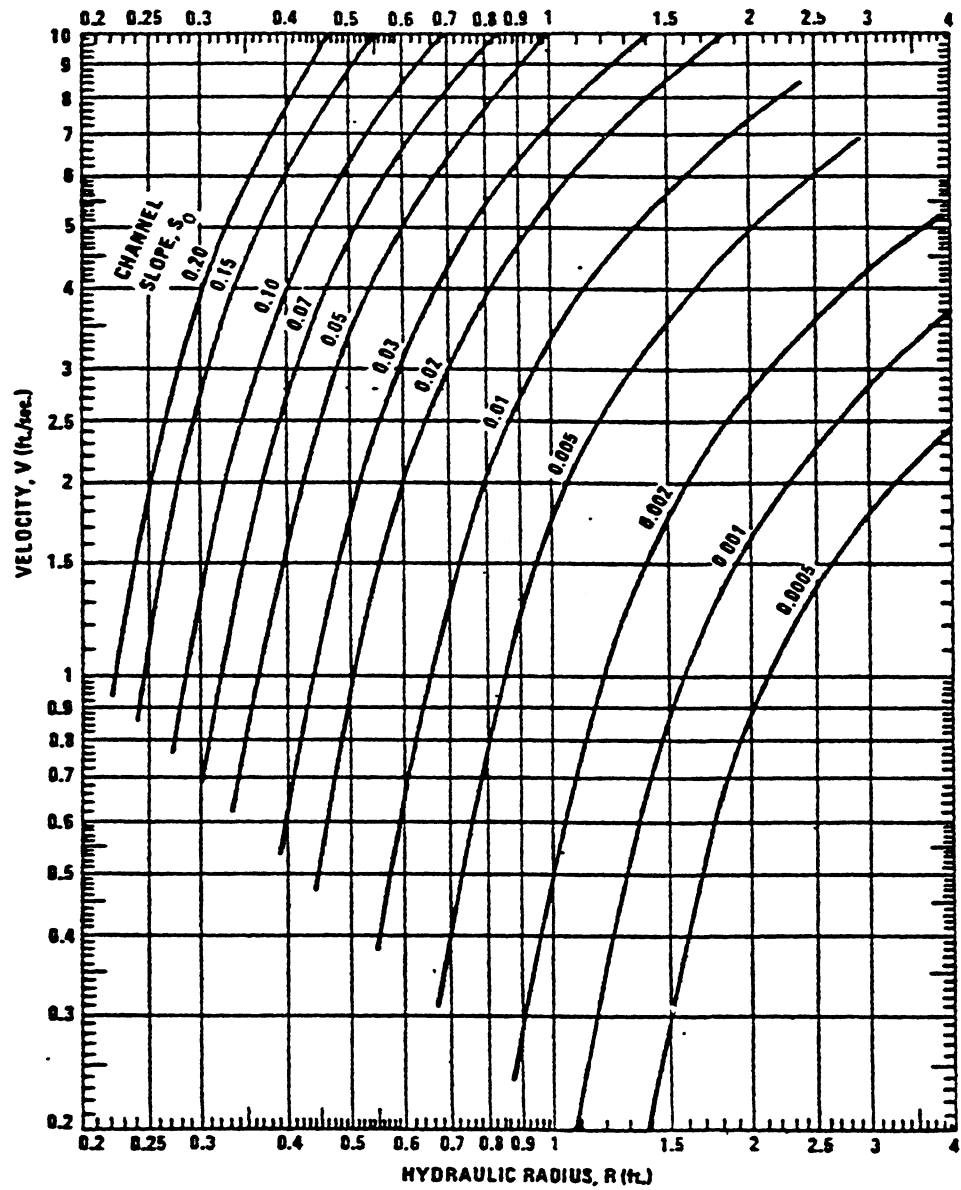


Figure 4 3 10
Flow Velocity for Channels Lined with Vegetation of Retardance C (Normann, 1975)

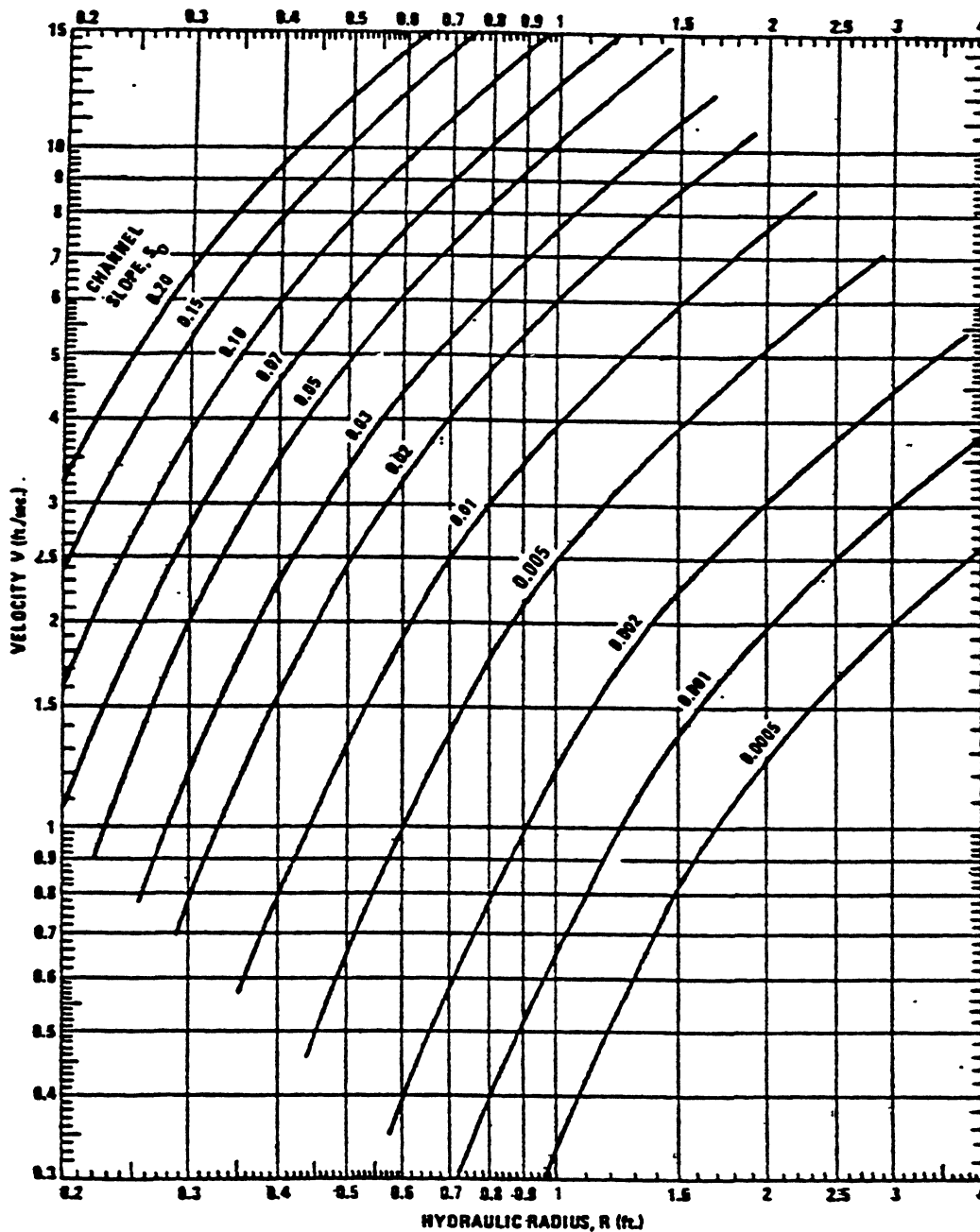


Figure 4.3 11
Flow Velocity for Channels Lined with Vegetation of Retardance D (Normann, 1975)

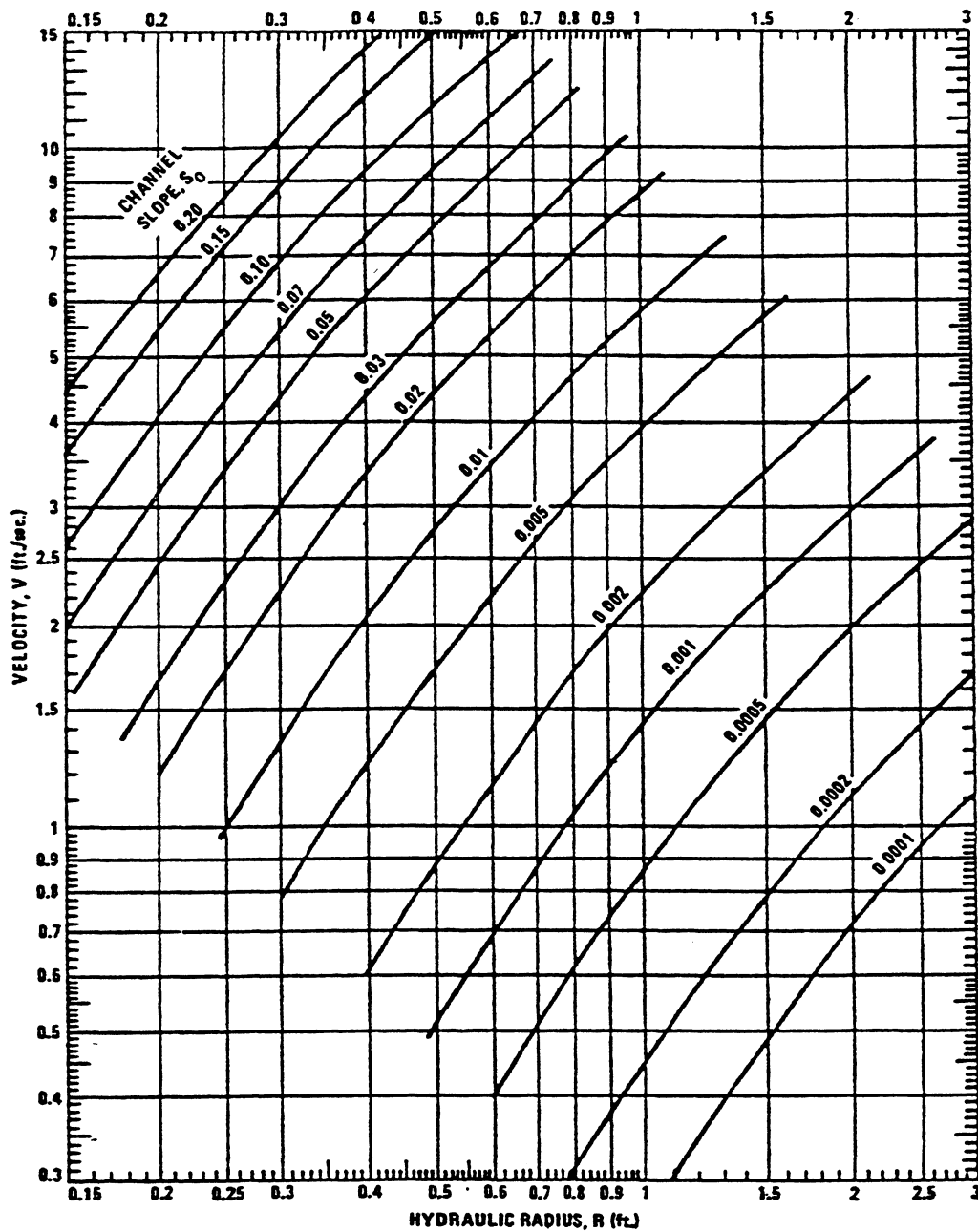
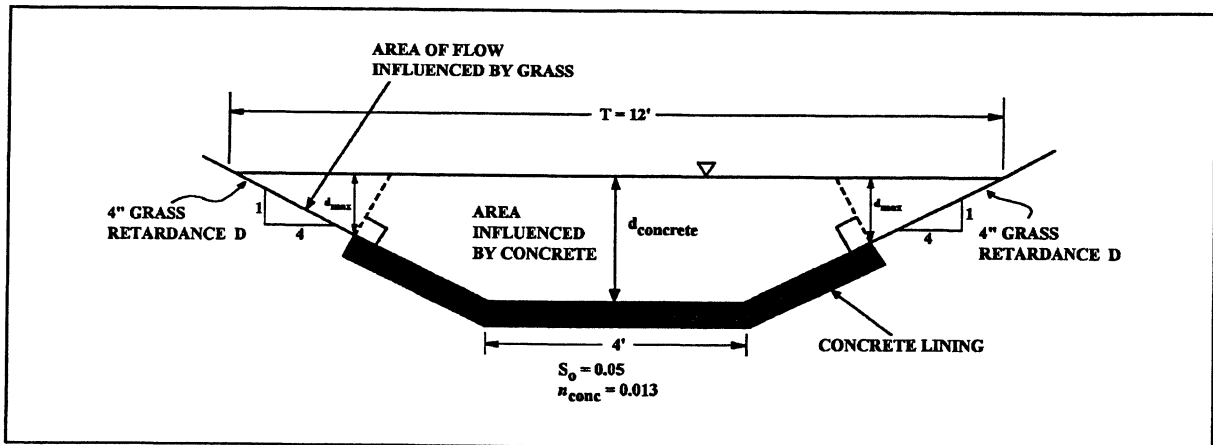


Figure 4 3 12
Flow Velocity for Channels Lined with Vegetation of Retardance E (Normann, 1975)

Example 4.3.2 (Normann, 1975)

Compute the capacity of the channel in Example 4.3.1, if the top width is limited to 12 feet, and the channel bottom and part of the sides is concrete lined. The channel slope is 0.05, the side slope is 4:1, and the bottom width is 4 feet. A grass mixture (retardance D) is to be an un-mowed, 4-inch length, on the average.



d_{\max} (grass mixture) = 0.45 ft (Figure 4.3.7) (Take the average value)

For $T_{\max} = 12$ ft, d (concrete) = 1.0 ft
 $A_{\text{total}} = (4.0)(1.0) + 4(1.0)^2 = 8.0 \text{ ft}^2$
 $P_{w \text{ total}} = 4.0 + 2(\sqrt{17})(1.0) = 12.25 \text{ ft}$

For shear boundary, take normal to side slopes at edge of concrete lining

$$A_{\text{grass}} = (4d_{\max} + 0.25d_{\max}^2)_{\text{grass}} * d_{\max \text{ grass}} = (4.25)(0.45)^2 = 0.86 \text{ ft}^2$$

$$P_{w \text{ grass}} = 2(\sqrt{17}) d_{\max} = 2(\sqrt{17})(.45) = 3.71 \text{ ft}$$

$$R_{\text{grass}} = (0.86 / 3.71) = 0.23 \text{ ft}$$

$$V_{\text{grass}} = 1.00 \text{ ft/sec (Figure 4.14)}$$

$$Q_{\text{grass}} = AV = (1.00)(0.86) = 0.86 \text{ cfs}$$

$$A_{\text{concrete}} = A_{\text{total}} - A_{\text{grass}} = 8.0 \text{ ft}^2 - 0.86 \text{ ft}^2 = 7.14 \text{ ft}^2$$

$$P_{w \text{ concrete}} = P_{w \text{ total}} - P_{w \text{ grass}} = 12.25 \text{ ft} - 3.71 = 8.54 \text{ ft}$$

$$R_{\text{concrete}} = 7.14 \text{ ft}^2 / 8.54 \text{ ft} = 0.84 \text{ ft}$$

$$Q_{\text{concrete}} = (1.49 / n) AR^{2/3} s_o^{1/2} = (1.49 / 0.013) (7.14) (0.84)^{2/3} (0.05)^{1/2}$$

$$Q_{\text{total}} = 163 \text{ cfs}$$

$$\text{Outlet velocity} = (Q_{\text{total}} / A_{\text{total}}) = 163 / 8.0 = 20.4 \text{ ft/sec}$$

4 3 5 - Design Procedures (after Normann, 1975)

A Lined, Non-Erodible Channels (See Example 4.3.1)

- Step 1 Perform the necessary hydrologic computations to establish the design discharge value (See Chapter 3)
- Step 2 Select a cross-sectional shape based on economics, aesthetics, safety, maintenance, etc. Select the appropriate n value from Table 4.2.1.
- Step 3 Calculate the section factor using Equation 4.2.2.
- Step 4 Determine the depth of flow using Figure 4.2.1. If this depth seems excessive, widen the channel section and determine a new depth of flow from Figure 4.2.1.
- Step 5 Check for the state of flow. Calculate the cross-sectional area of flow and hydraulic depth of flow (See Table 4.2.1). Calculate $V = Q/A$. Calculate the Froude number from $F = V/\sqrt{gD}$. If this Froude number is equal to or greater than one, then critical or supercritical flow exists and trouble could be experienced with standing waves, flow around bends, and general flow instability. Another cross-sectional shape or the use of check structures should be considered.
- Step 6 Modify the final section dimensions for practicality and add a freeboard depth to the channel.

B Erodible Channels

- Step 1 Perform hydrologic computations.
- Step 2 Select design flows for permanent lining material and for temporary linings based on subjective risk analyses.
- Step 3 Define soil erodibility
- The difficulties involved in defining the erodibility of cohesive soils is well described by Partheniades (1971), based on the MSU work, which covered ten (10) soils of different characteristics.
 - Soils with a gravel, sand and clay mixture are erosion resistant; fine-grained sands or silts are erodible; while plastic and semi-plastic soils are in the intermediate range.
 - The soil erodibility factor for the Universal Soil Loss Equation, developed by Wischmeier and Smith (1978), can also be used as a guide to soil erodibility. If the designer has no knowledge of the erodibility of the soil at a particular channel site, a reasonable estimate of d_{max} may be obtained by interpolating halfway between the "erosion resistant" and "erodible" lines of the maximum permissible depth charts (Figures 4 3.4 and 4.3 7).

- Step 4: Define channel shape, slope and maximum top width and lining.
- For purposes of safety, construction, maintenance, and erosion resistance, it is suggested that the channel side slopes be kept as flat as possible. Ideally, side slopes should be 3:1 or flatter for erosion resistance. Flatter slopes may be necessary for safety or other reasons.
- Step 5: Determine d_{\max} for the selected slope, and soil erodibility (Figure 4.3.4 or 4.3.8).
- Step 6: Determine hydraulic radius (R) and area (A) for the selected channel geometry and d_{\max} (Table 4.1).
- Step 7: Determine the velocity of flow using:

- (a) For rock rip-rap: Use Manning Formula with Roughness Coefficient determined by

$$n = 0.0395 D_{50}^{1/6} \quad (4.3.5)$$

where D_{50} = mean stone size (feet).

- (b) For bare soil: Use

$$V = 22.81 R^{0.591} S_0^{0.286} \quad (4.3.6)$$

- (c) For grass lined channels: Use appropriate chart of Figures 4.3.8 through 4.3.12.

Step 8: Determine $Q = VA$

If Q does not satisfy the design Q , select another channel size and return to Step 4 or select another lining material and return to Step 5. Also, consider the feasibility of additional inlets to reduce Q .

A computation sheet, shown in Figure 4.3.13 has been developed to facilitate the foregoing design procedure (Normann, 1975). In addition a nomograph for determining the geometric properties of trapezoidal channels is presented in Figure 4.3.14.

PROJECT: _____ **DATE:** _____
STATION: _____ **TO STATION:** _____ **DESIGNER:** _____
DRAINAGE AREA = _____ **ACRES** **CHECKED BY:** _____
DATE: _____
HYDROLOGIC COMPUTATIONS:

$$S_0 = \underline{\hspace{10cm}}$$
[illegible]

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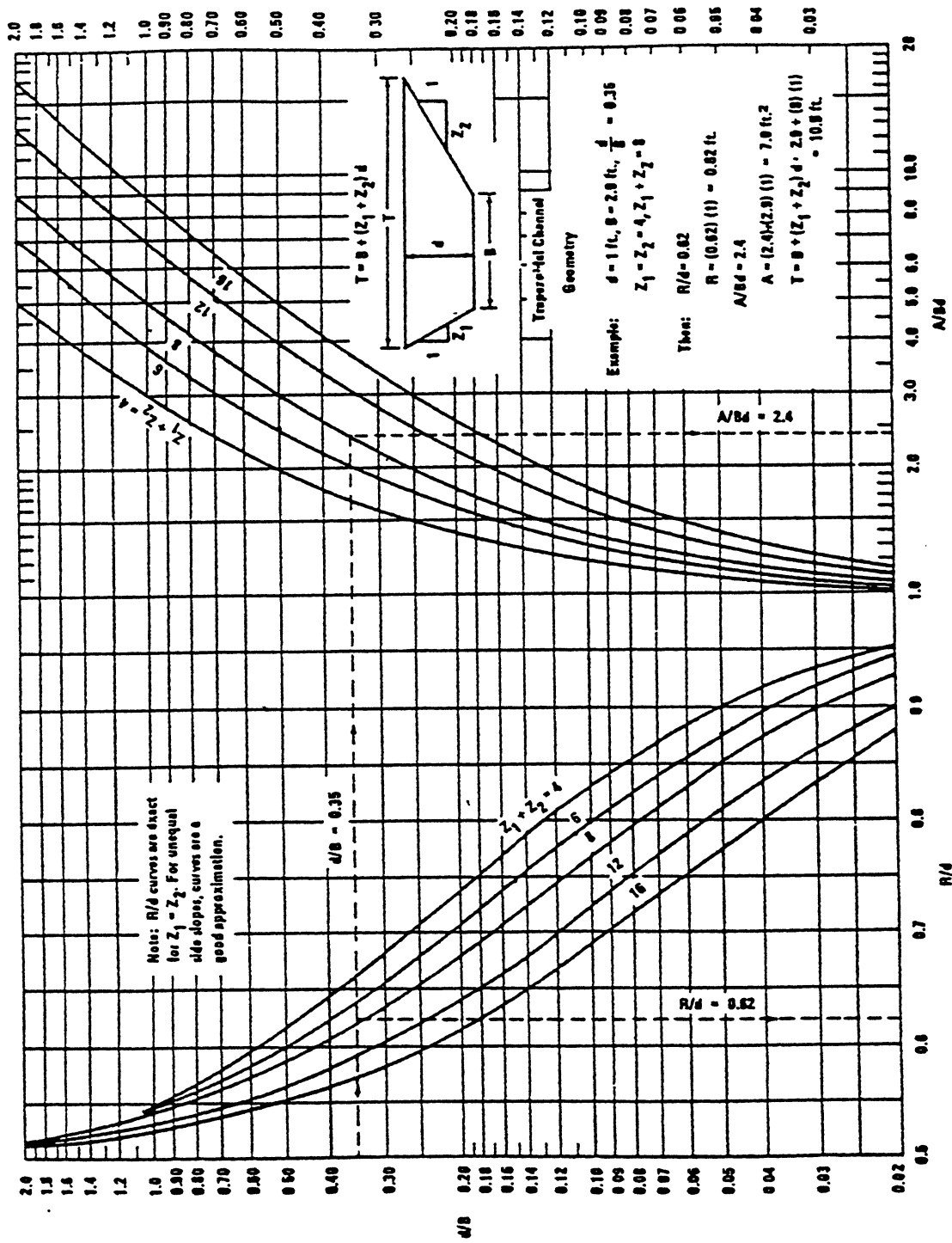


Figure 4.3.14
 Nomograph for Design of Trapezoidal Channel Section (Normann, 1975)

4.3.6 Flow in Bends

Flow around a bend in an open channel creates currents which impose higher shear stresses on the channel sides and bottom. According to Anderson (1973), the location of the maximum shear varies depending on the position in the bend.

Figure 4.3 15 has been prepared to adjust the D_{50} of rock rip-rap lining for the higher shear stresses in a bend, depending on the surface width of the channel ($B_s = T$) and the mean radius of the bend (R_O). To use the chart, determine K_3 based on the ratio B_s/R_O . Then, multiply the D_{50} , determined for the straight channel reaches, by K_3 to obtain a larger stone size for use in the bend. For instance, assume that a channel with a top width of 12 feet requires a stone size of $D_{50} = 0.5$ feet in the straight reaches. Assume that the channel has a bend with a 24 ft. radius. Then $B_s/R_O = 12/24 = 0.5$ from Figure 4.18, for $B_s/R_O = 0.5$, $K_3 = 1.18$

$$(D_{50})_{Bend} = 1.18(D_{50})_{Straight} = (1.18)(0.5) = 0.59 \text{ ft}$$

Therefore, the stone in the bend area should have a D_{50} of 0.59 ft. Since it is not possible to predict the location of the maximum shear, the entire channel cross-section must be protected with the same stone.

Figure 4.3 15 also indicates the B_s/R_O values requiring additional protection with other lining materials. It appears that as long as B_s/R_O is less than 0.4, K_3 is less than 1.1 and no additional protection is necessary. When B_s/R_O is greater than 0.4, the lining in the bend area should be reinforced with rip-rap.

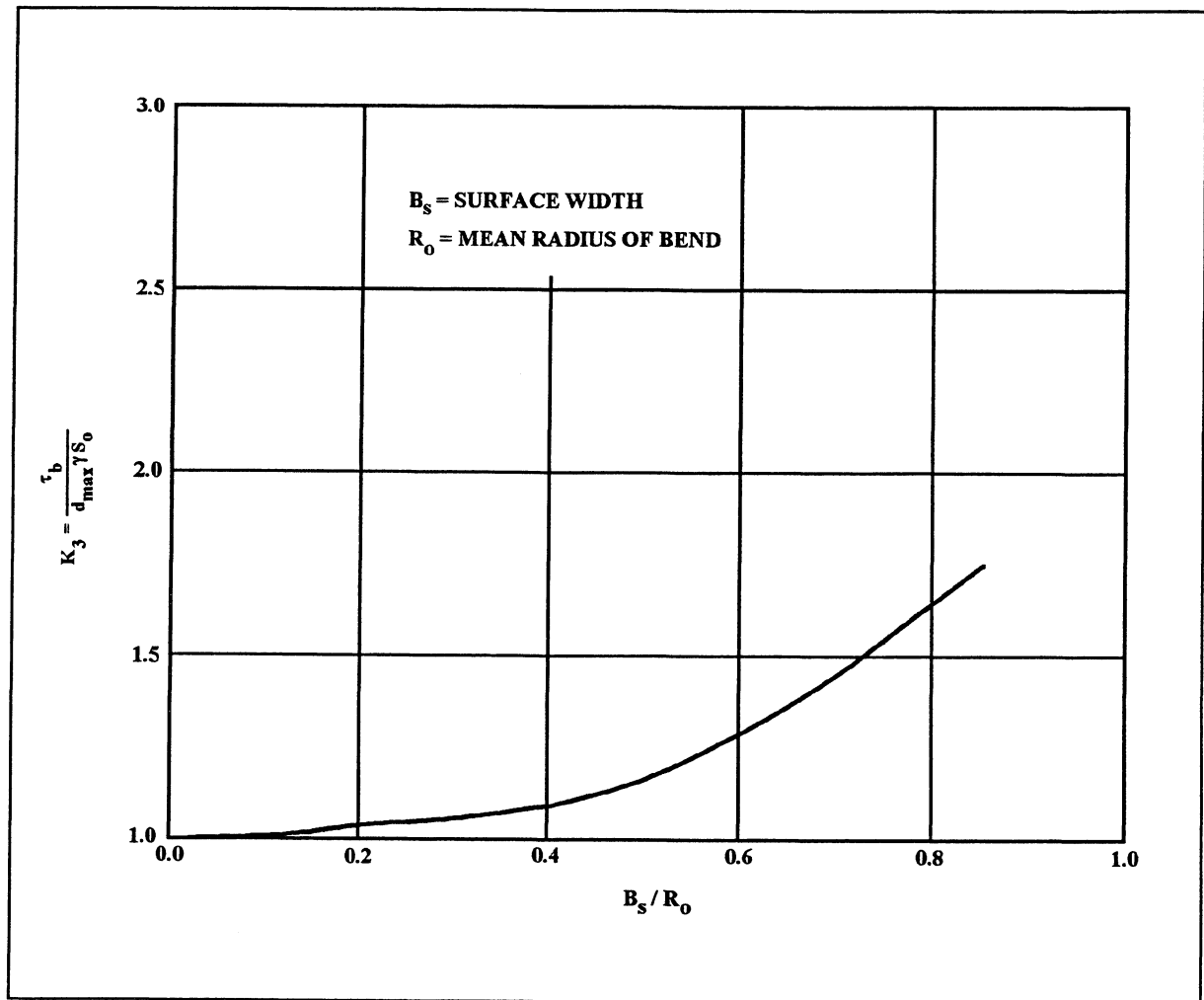


Figure 4.3.15
 Ratio of Maximum Boundary Shear in Bends to Maximum
 Bottom Shear in Straight Reaches (Anderson, 1973)
 Note $B_s = T$

4 4 - Gradually Varied Flow

Gradually varied flow is defined as steady flow which has a gradual change in depth along the channel length. This is typical of many natural streams or channels. The analysis and computation of this type of flow is quite complex. This section will develop the basic theory and give a simplified design procedure used in the analysis of this flow.

The basic assumptions for gradually-varied-flow theory presented here are:

- 1 The head loss for a given section is identical of that for uniform flow with the same velocity and section hydraulic radius.
- 2 The channel has a constant cross-section.
- 3 The Manning roughness coefficient n is constant for the length of the channel under consideration and does not vary with depth.
- 4 The depth of flow is the same as the distance normal to the free surface.

4 4.1 - Development of Analysis

Recalling Equation 4 1 1 and Figure 4 1 1, the total energy at a given section may be written as

$$H = \frac{V^2}{2g} + Z + d \cos\theta \quad (4.4 1)$$

By differentiating the above equation with respect to x , the slope of the energy grade line dH/dx is given as S_f . If the slope decreases, the values are negative; if the slope increases, the values are positive. Therefore, Equation 4 4.1 may be simplified by solving for the water surface slope dd / dx .

$$\frac{dd}{dx} = \frac{S_o - S_f}{\cos\theta + \frac{d(V^2/2g)}{dd}} \quad (4.4.2)$$

For small slopes a $\cos\theta = 1$ and $dd / dx = dy / dx$ which gives

$$\frac{dy}{dx} = \frac{S_o - S_f}{1 + d(V^2/2g)dy} \quad (4 4.3)$$

This is the general equation for gradually varied flow. It is easily seen that for uniform flow $S_o = S_f$ and $dy / dx = 0$. When $S_o > S_f$ dy / dx is positive, and when $S_o < S_f$ dy / dx is negative.

Table 4.4.1 lists the many possible types of flow profiles which may occur in open channels and their classification. Figure 4.4.1 presents these conditions graphically. In engineering design the most typical application of the gradually varied flow equation is backwater.

Backwater occurs as a result of a tributary stream which may have a lower depth than the larger channel to which it contributes. The result is a subsequent backing up of the tributary and a change in the water surface profile. The depth and extent of this backwater is needed to provide the planner and engineer with data to insure that no property or lives are endangered by the increased depths.

There are many detailed computer programs which have been developed to analyze this effect which have been shown to produce reliable results. HEC-2 by the Army Corps of Engineers (COE, 1990), WSP-2 by the Soil Conservation Service (USDA, 1987) or WSPRO by the United States Geologic Survey and Federal Highway Administration (USGS and FHWA, 1988) are recommended, if available. However, if a small channel is being analyzed or a rough estimate is desired, the following section presents a method which may be used.

Table 4.4.1
Types of Flow Profiles in Prismatic Channels (Chow, 1988)

Channel slope	Designation			Relation of y to y_n and y_c			General type of curve	Type of flow
	Zone 1	Zone 2	Zone 3	Zone 1	Zone 2	Zone 3		
Horizontal $S_0 = 0$	None			$y > y_n > y_c$			None	None
		H2		$y_n > y > y_c$			Drawdown	Subcritical
			H3	$y_n > y_c > y$			Backwater	Supercritical
Mild $0 < S_0 < S_c$	M1			$y > y_n > y_c$			Backwater	Subcritical
		M2		$y_n > y > y_c$			Drawdown	Subcritical
			M3	$y_n > y_c > y$			Backwater	Supercritical
Critical $S_0 = S_c > 0$	C1			$y > y_c = y_n$			Backwater	Subcritical
		C2		$y_c = y = y_n$			Parallel to channel bottom	Uniform-critical
			C3	$y_c = y_n > y$			Backwater	Supercritical
Steep $S_0 > S_c > 0$	S1			$y > y_c > y_n$			Backwater	Subcritical
		S2		$y_c > y > y_n$			Drawdown	Supercritical
			S3	$y_c > y_n > y$			Backwater	Supercritical
Adverse $S_0 < 0$	None			$y > (y_n)^* > y_c$			None	None
		A2		$(y_n)^* > y > y_c$			Drawdown	Subcritical
			A3	$(y_n)^* > y_c > y$			Backwater	Supercritical

* y_n in parentheses is assumed a positive value.

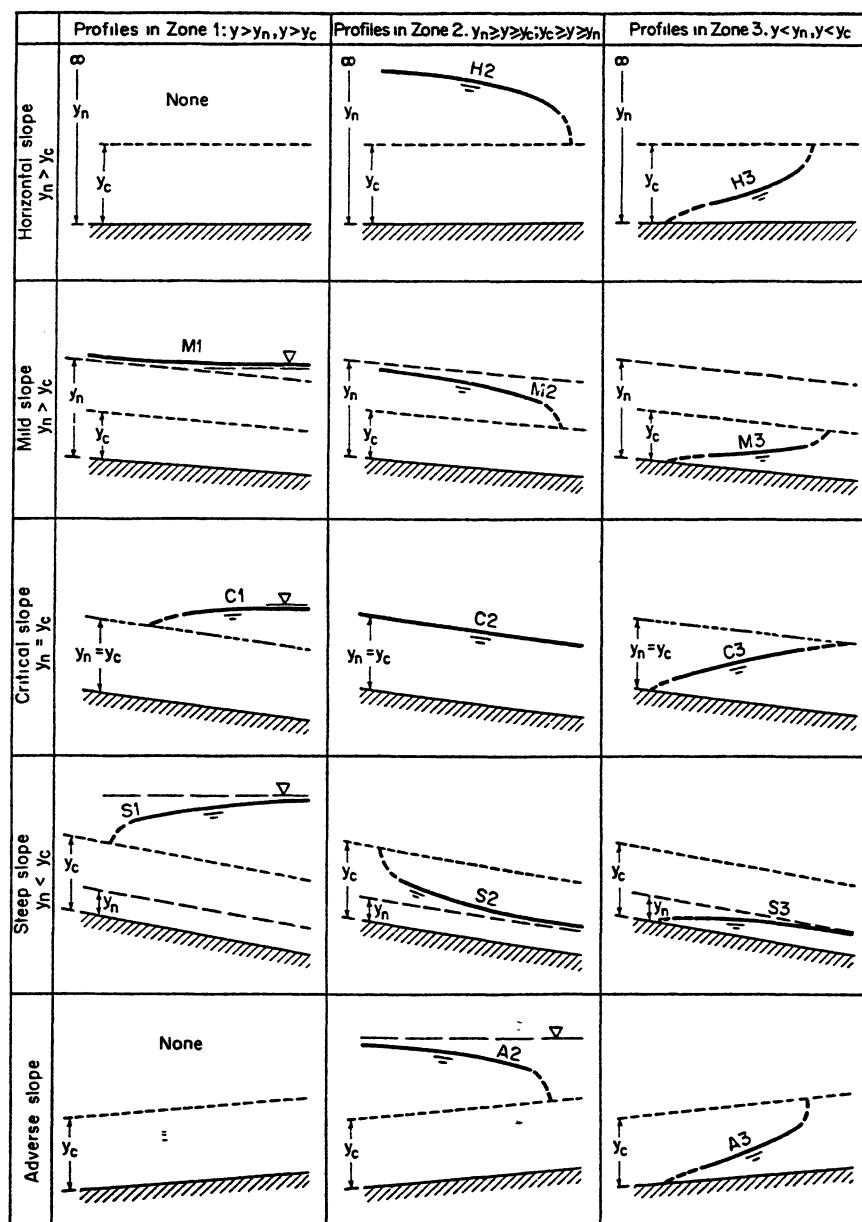


Figure 4.4.1
Classification of Flow Profiles of Gradually Varied Flow (Chow, 1988)

4.4.2 - Backwater Curve Calculations

The method presented here is the direct step method. This simple method breaks the channel into short reaches and the computations are done from step to step, and the distance to specified depths determined. This method may be applied to prismatic channels. For more sophisticated manual methods the reader is referred to Chow (1988).

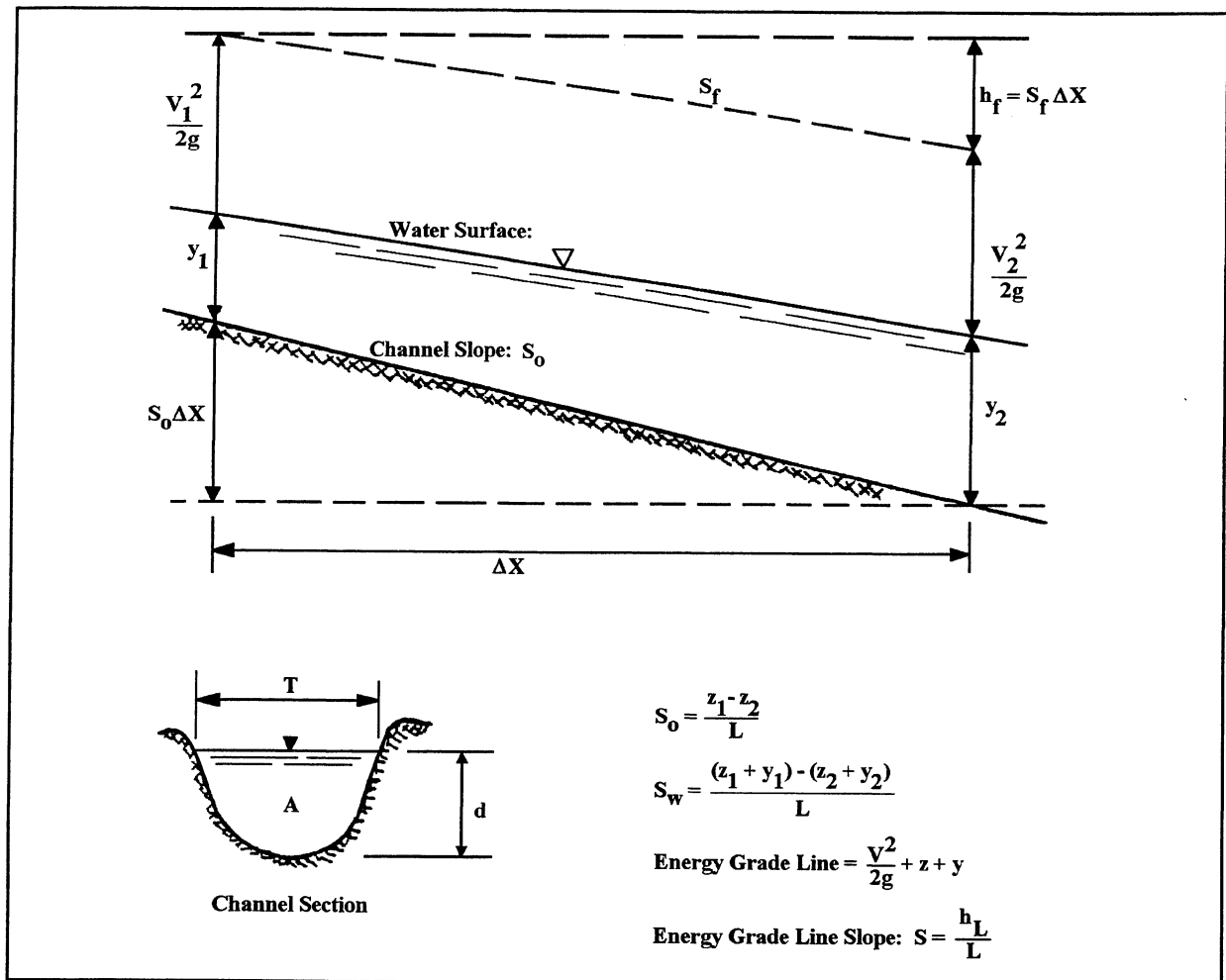


Figure 4.4.2
A Channel Length Used for the Development of the Direct Step Method

From Figure 4.4.2 the following expression which relates the energy head in a reach of length ΔX , can be developed.

$$S_o \Delta X + y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} + S_f \Delta X \quad (4.4.4)$$

Since the specific energy E has been defined as $y + V^2/2g$ we can solve Equation (4.4.2) for the increment of length ΔX .

$$\Delta X = \frac{E_2 - E_1}{S_o - S_f} = \frac{\Delta E}{S_o - S_f} \quad (4.4.5)$$

S_o is found from channel characteristics and S_f is determined from the Mannings' Formula:

$$S_f = \frac{n^2 V^2}{2.22 R^{4/3}} \quad (4.4.6)$$

The procedure used in determining the water surface profile is listed below:

- Step 1: With a starting depth y , which is usually a known condition, proceed upstream in equal depth increments.
- Step 2: Compute the area and the hydraulic radius for the given depth y .
- Step 3: Compute the velocity by dividing the flow rate (given) by the flow area and calculate the velocity head ($V^2/2g$).
- Step 4: Determine the specific energy, $E = (y + V^2/2g)$, and change in specific energy (ΔE) from the previous step.
- Step 5: Compute S_f from Equation 4.4.6.
- Step 6: Compute the average friction slope between the steps using an arithmetic average.
- Step 7: Compute the difference between the channel slope and the average friction slope.
- Step 8: Compute the length between the sections by Equation 4.4.5.

Example 4.4.1 outlines the above procedure.

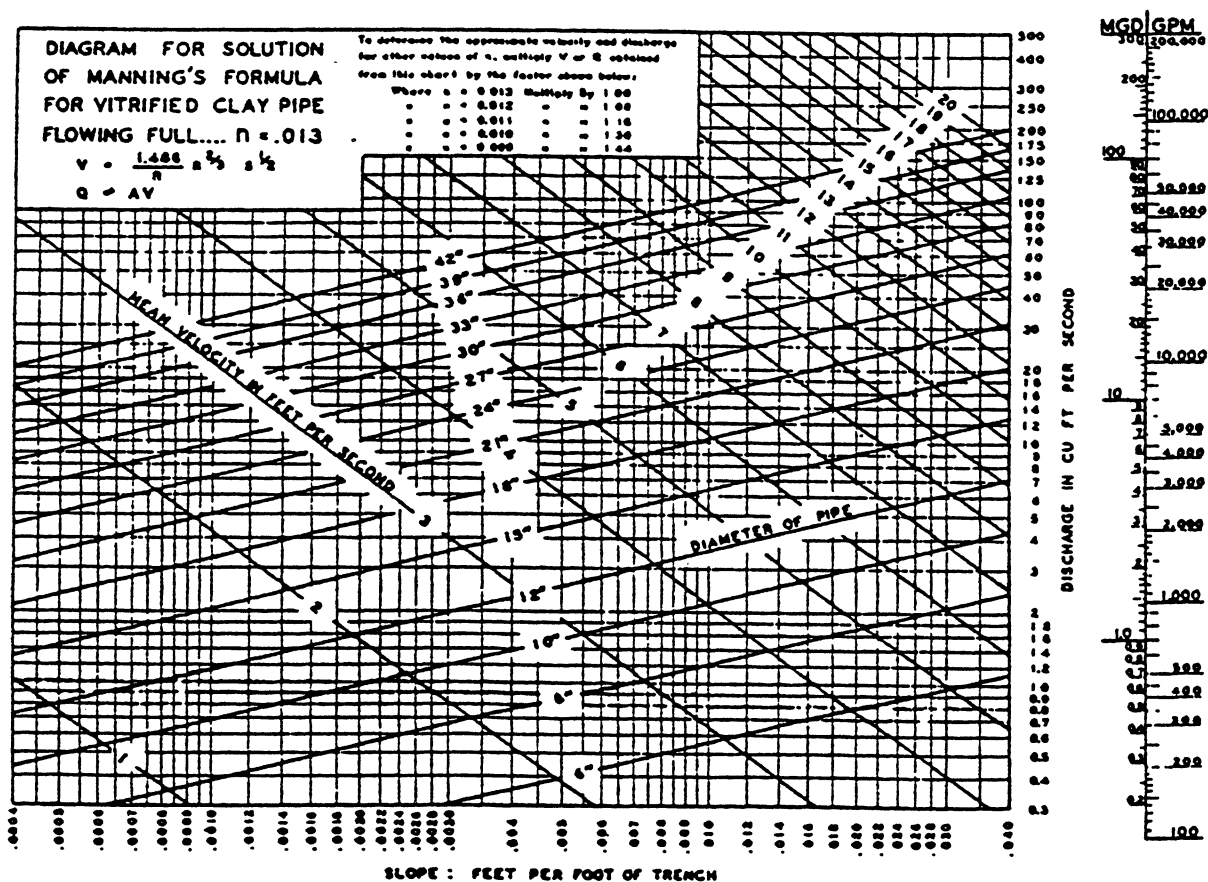


Figure 4.4.3
 Graphical Solution of the Manning Equation for Circular Pipe Flowing Full

Example 4.4.1

For a rectangular concrete channel with a ten foot width, flow rate of 360 cfs, Manning n of 0.015 and a slope of 0.0023 ft/ft, determine the backwater profile. The downstream depth is 5 feet. Work upstream in 0.1 ft increments until a depth of 4.4 ft is reached.

The computations are shown on the next page. For each depth, the area, hydraulic radius, velocity and velocity head may be determined. The energy, change in energy, friction slope and the difference between the bed slope and the friction slope also can be calculated. Finally, the incremental reach ΔX and the composite length x are found. For this example, the depth of 4.4 feet, occurs 893 feet upstream.

Y	A	R	$R^{4/3}$	V	$V^2/2g$	E	ΔE	S_f	\bar{S}_f	$S_o - S_f$	ΔX	X
(ft)	(ft ²)	(ft)	(ft ^{4/3})	(ft/sec)	(ft)	(ft)	(ft)	(ft/ft)	(ft/ft)	(ft/ft)	(ft)	(ft)
5.0	50	2.50	3.39	7.20	0.80	5.80	-	0.00155	-	-	-	-
4.9	49	2.47	3.35	7.35	0.84	5.74	0.06	0.00163	0.00159	0.00071	84.50	84.50
4.8	48	2.45	3.30	7.50	0.87	5.67	0.07	0.00173	0.00168	0.00062	112.90	197.40
4.7	47	2.42	3.25	7.66	0.91	5.61	0.06	0.00183	0.00178	0.00052	115.36	312.76
4.6	46	2.40	3.20	7.83	0.95	5.55	0.06	0.00194	0.00189	0.00041	144.89	457.65
4.5	45	2.37	3.16	8.00	0.99	5.49	0.05	0.00205	0.00200	0.00030	164.66	622.31
4.4	44	2.34	3.11	8.18	1.04	5.44	0.05	0.00218	0.00212	0.00018	270.71	893.02

4.5 - HY-8 PROGRAM

The Federal Highway Administration (FHWA) has developed analytical and empirical techniques for the hydraulic analysis and design of culverts. The design engineer may utilize the FHWA publications to analyze culverts for a single design discharge and, with some additional effort, develop a culvert performance curve. In addition, these techniques allow the consideration of inflow and outflow hydrographs, storage and routing, and energy dissipation. Different scenarios require considerable effort to compute by hand. The HY-8 program was developed to quickly and accurately solve these culvert system techniques.

A culvert system is composed of the actual hydraulic structure, as well as hydrologic inputs, storage and routing considerations, and energy dissipation devices and strategies. HY-8 automates the methods presented in HDS 5, "Hydraulic Design of Highway Culverts", HEC-14, "Hydraulic Design of Energy Dissipators for Culverts and Channels", HEC-19, "Hydrology", and information published by pipe manufacturers pertaining to the culvert sizes and materials. HY-8 is composed of the four previously named programs: culvert analysis, hydrograph generation, routing, and energy dissipation. A full description of the program and its capabilities may be obtained by consulting the FHWA references at the end of this chapter.

The current (July, 1995) version of HY-8 is 4.1. A version 4.3 which was included in Version 5 of HYDRAIN (Young and Krolak, 1992) had problems with shapes that were not regular and passing data to energy.exe. A version 5 of HY8 will not be distributed. The next version will be 6 which will be distributed with HYDRAIN 6 late this year. Version 6 of HYDRAIN and HY8 will be able to work in either SI or english units. The HY-8 program may be obtained from the National Technical Information Service, Springfield, Virginia, 22161 or from McTrans by calling (800) 226-1013.

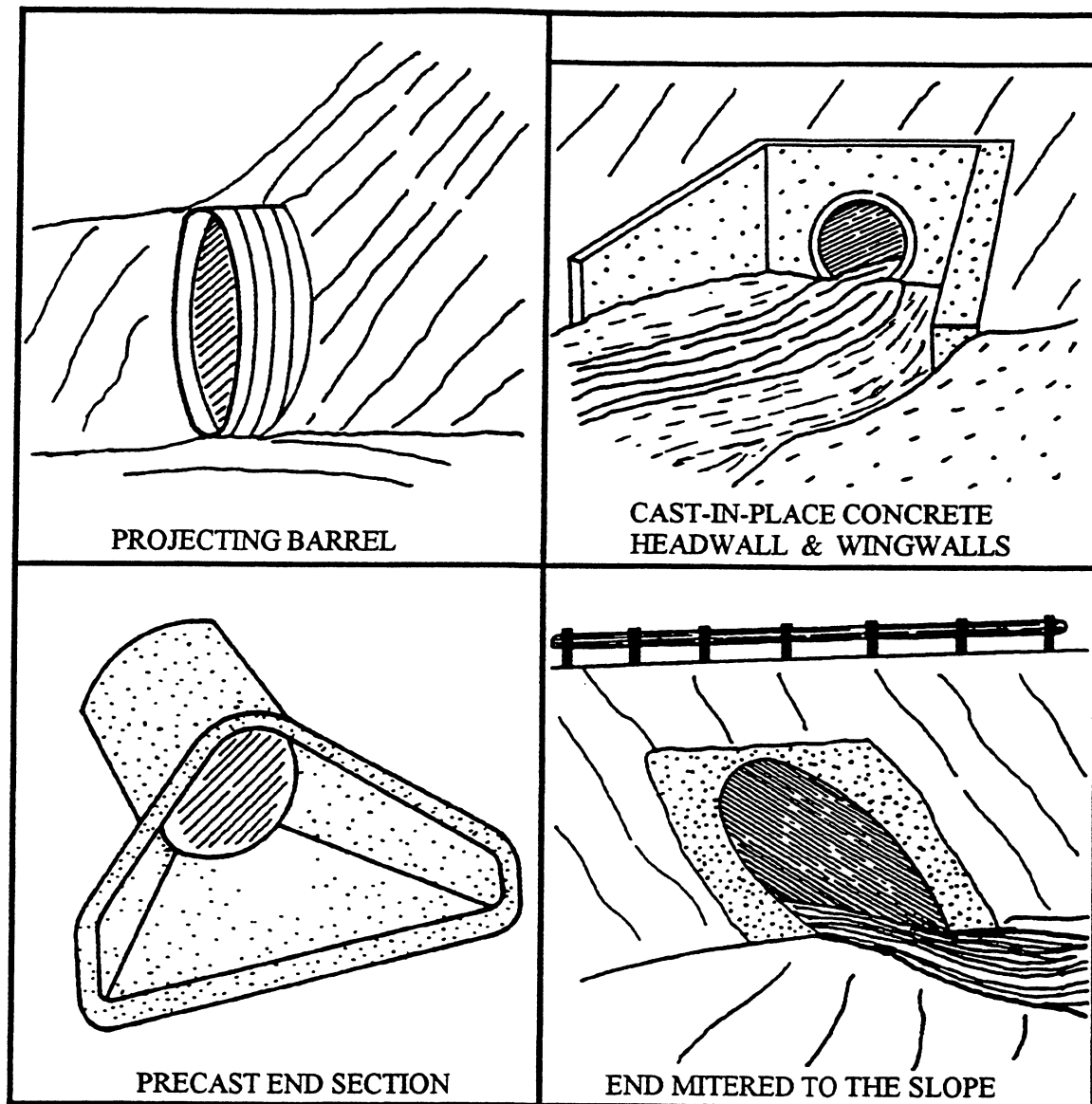


Figure 4.5.1
Four Standard Inlet Types (schematic) (FHWA, 1985)

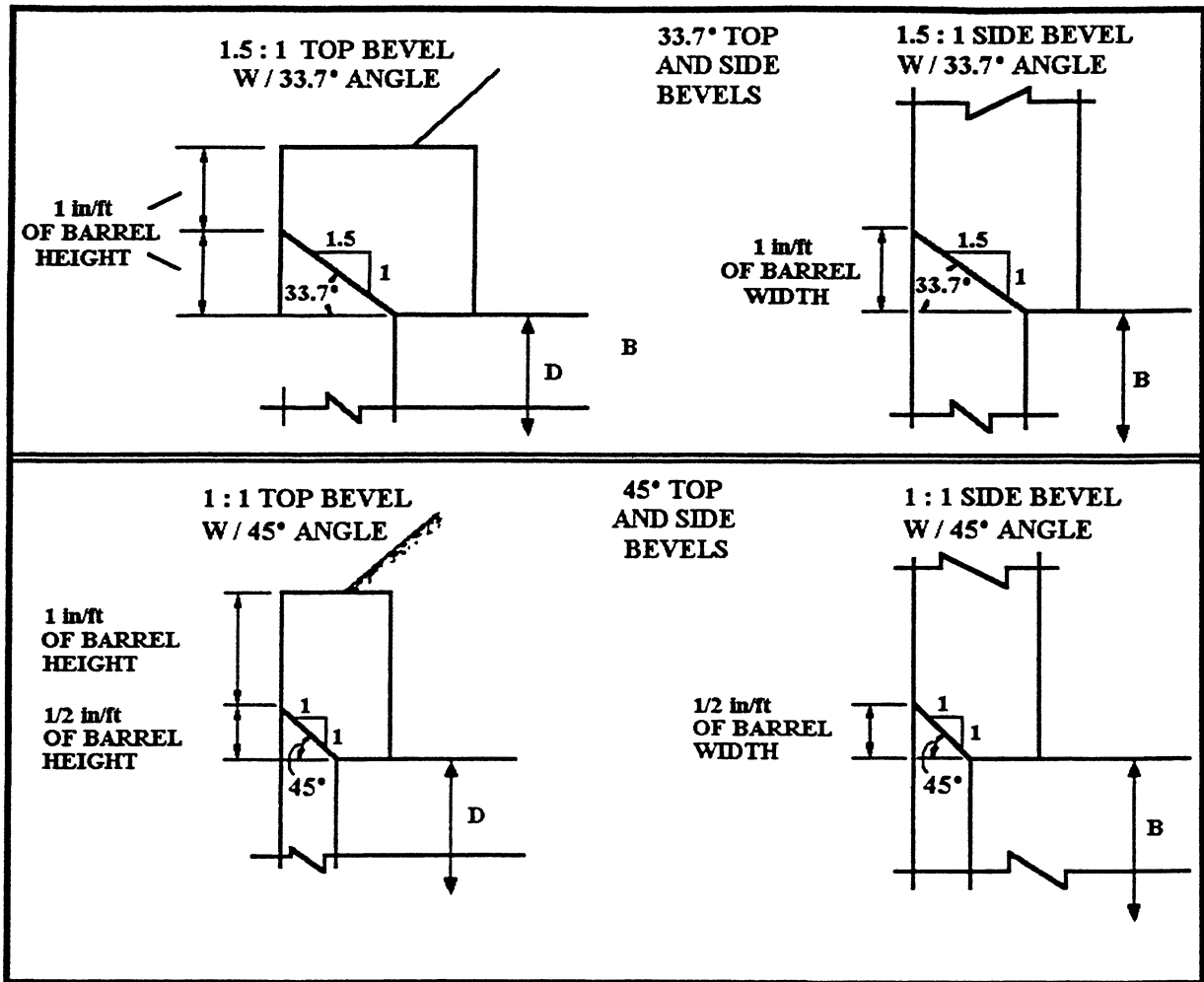


Figure 4.5.2
Beveled Edge Inlets (FHWA, 1985)

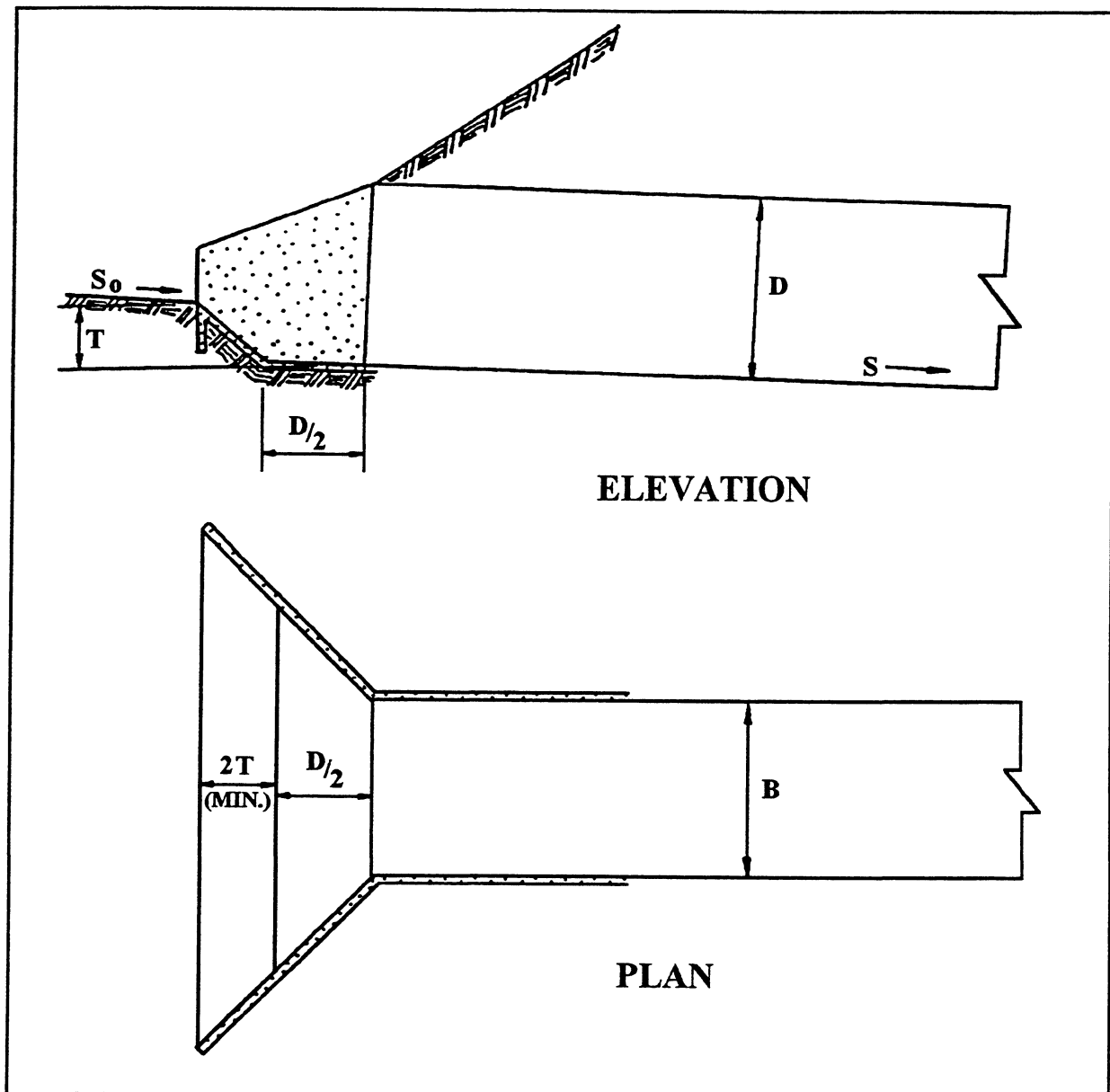


Figure 4.5.3
Culvert with Depressed Apron and Wingwalls (FHWA, 1985)

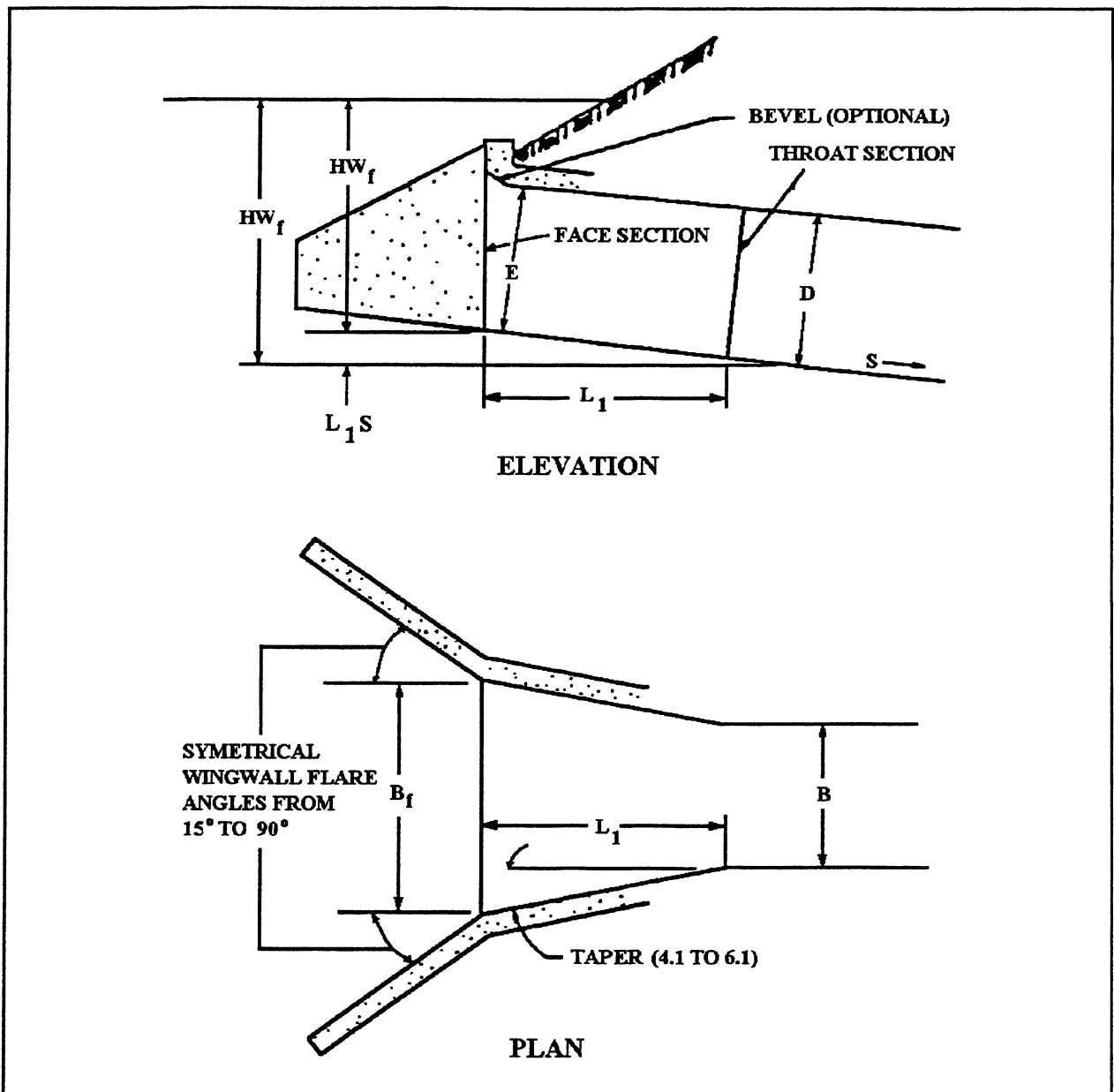


Figure 4.5.4
Side-tapered Inlet (FHWA, 1985)

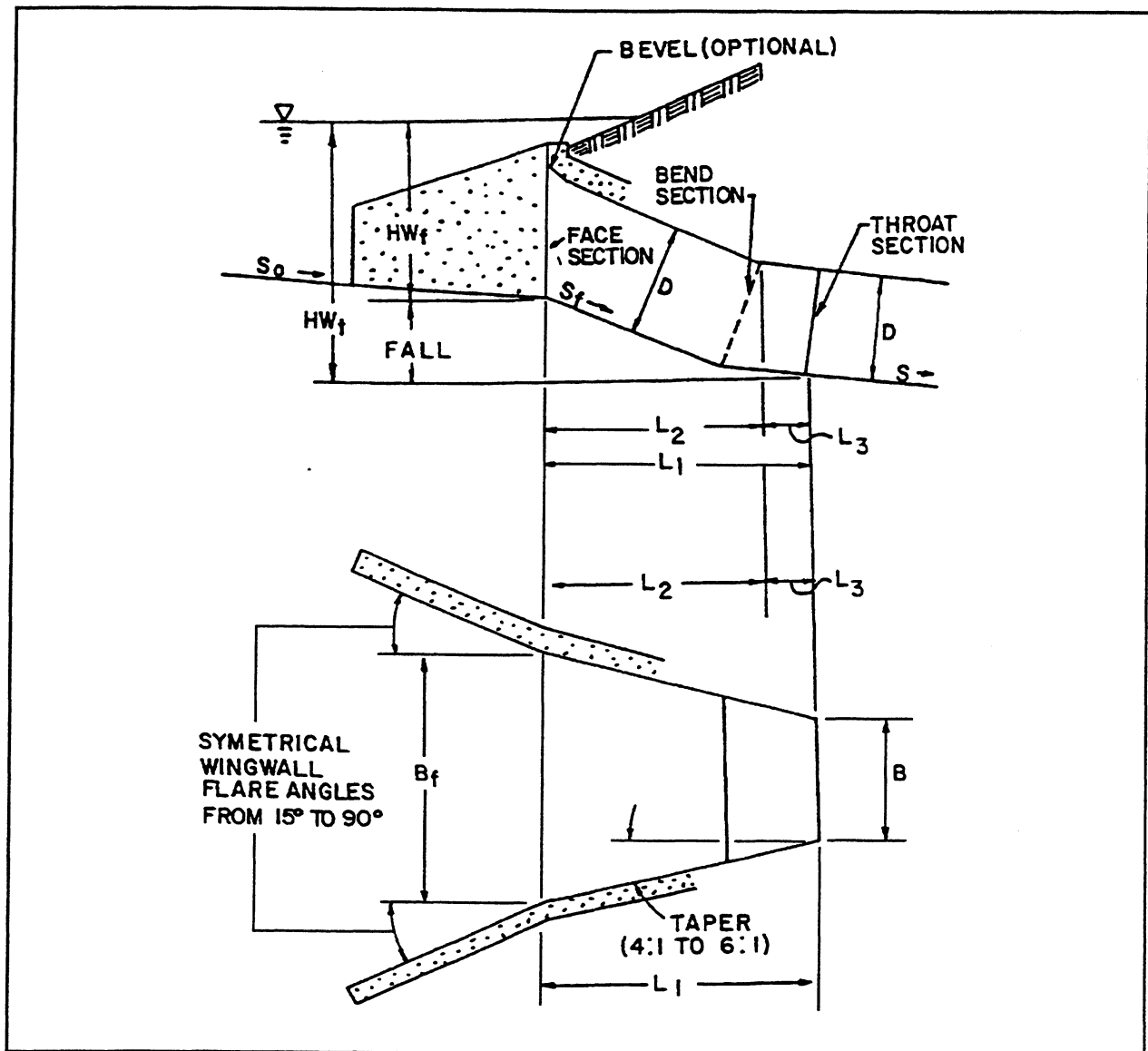


Figure 4.5.5
Slope-tapered Inlet with Vertical Face (FHWA, 1985)

Example 4.5.1

This example illustrates the use of the HY-8 computer program for developing an elevation-discharge curve resulting from a twin culvert.

Using 48 feet long twin 38-inch by 66-inch CMP culverts, determine the elevation-discharge relationship at the culvert outlet given a downstream and roadway cross-section shown below. The following additional data describes the twin culverts and is used for input for the HY-8 computations:

Invert Elevation	654.17 ft.	Outlet Elevation	654.16 ft.
Culvert Length	212.0 ft.	Inlet Type	Conventional
Inlet Edge and Wall	Projecting	Inlet Depression	None

In addition, the roadway elevations in the vicinity of the culverts are entered into the program using x, y coordinates (see program output, page 55).

The following pages contain the HY-8 results and a summary of the input. There is no overtopping for the culvert flows ranging between 0 and 50 cfs. To find the culvert flow at which overtopping would occur, the program may be run again with higher flows. As a result, it was determined that around 185 cfs the flow begins to overtop the roadway. The elevation-discharge is shown on the next page. As an example, at elevation 656.23 feet the total flow at the outlet of the culvert is 25 cfs.

CURRENT DATE: 08-12-1994
CURRENT TIME: 09:10:57

FILE DATE: 08-11-1994
FILE NAME: EX45-1

FHWA CULVERT ANALYSIS
HY-8, VERSION 4.0

C U L V	SITE DATA			CULVERT SHAPE, MATERIAL, INLET				
	INLET ELEV. (FT)	OUTLET ELEV. (FT)	CULVERT LENGTH (FT)	BARRELS SHAPE MATERIAL	SPAN (FT)	RISE (FT)	MANNING n	INLET TYPE
1	654.17	654.16	212.00	2 CMPA	5.42	3.33	.024	CONVENTIONAL
2								
3								
4								
5								
6								

SUMMARY OF CULVERT FLOWS (CFS)				FILE: EX45-1				DATE: 08-11-1994		
ELEV (FT)	TOTAL	1	2	3	4	5	6	ROADWAY	ITR	
654.17	0	0	0	0	0	0	0	0	0	1
654.75	5	5	0	0	0	0	0	0	0	1
655.08	10	10	0	0	0	0	0	0	0	1
656.10	15	15	0	0	0	0	0	0	0	1
656.16	20	20	0	0	0	0	0	0	0	1
656.23	25	25	0	0	0	0	0	0	0	1
656.29	30	30	0	0	0	0	0	0	0	1
656.36	35	35	0	0	0	0	0	0	0	1
656.43	40	40	0	0	0	0	0	0	0	1
656.51	45	45	0	0	0	0	0	0	0	1
656.59	50	50	0	0	0	0	0	0	0	1
660.50	185	185	0	0	0	0	0	0	0	OVERTOPPING

SUMMARY OF ITERATIVE SOLUTION ERRORS				FILE: EX45-1		DATE: 08-11-1994	
HEAD ELEV(FT)	HEAD ERROR(FT)	TOTAL FLOW(CFS)	FLOW ERROR(CFS)	% FLOW ERROR			
654.17	0.00	0	0	0.00			
654.75	0.00	5	0	0.00			
655.08	0.00	10	0	0.00			
656.10	0.00	15	0	0.00			
656.16	0.00	20	0	0.00			
656.23	0.00	25	0	0.00			
656.29	0.00	30	0	0.00			
656.36	0.00	35	0	0.00			
656.43	0.00	40	0	0.00			
656.51	0.00	45	0	0.00			
656.59	0.00	50	0	0.00			
<1> TOLERANCE (FT) = 0.010				<2> TOLERANCE (%) = 1.000			

CURRENT DATE: 08-12-1994
CURRENT TIME: 09:10:57

FILE DATE: 08-11-1994
FILE NAME: EX45-1

PERFORMANCE CURVE FOR CULVERT # 1 - 2 (5.416667 BY 3.333333) CMPA

DIS- CHARGE FLOW (cfs)	HEAD- WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRITICAL DEPTH (ft)	OUTLET VEL. (fps)	OUTLET DEPTH (ft)	TAILWATER VEL. (fps)	TAILWATER DEPTH (ft)
0	654.17	0.00	0.00	0-NF	0.00	0.00	0.00	0.00	0.00	-0.21
5	654.75	0.38	0.58	3-M2t	1.31	0.26	2.15	0.35	0.54	0.35
10	655.08	0.67	0.91	3-M2t	2.17	0.40	2.37	0.52	0.63	0.52
15	656.10	0.83	1.93	6-FFn	3.33	0.50	0.53	3.33	0.70	0.65
20	656.16	0.98	1.99	6-FFn	3.33	0.59	0.70	3.33	0.75	0.75
25	656.23	1.11	2.06	6-FFn	3.33	0.68	0.88	3.33	0.79	0.84
30	656.29	1.24	2.12	6-FFn	3.33	0.74	1.05	3.33	0.85	0.91
35	656.36	1.36	2.19	6-FFn	3.33	0.81	1.23	3.33	0.90	0.97
40	656.43	1.48	2.26	6-FFn	3.33	0.88	1.41	3.33	0.94	1.02
45	656.51	1.58	2.34	6-FFn	3.33	0.94	1.58	3.33	0.98	1.07
50	656.59	1.69	2.42	6-FFn	3.33	1.01	1.76	3.33	1.02	1.12
El. inlet face invert					654.17 ft	El. outlet invert		654.16 ft		
El. inlet throat invert					0.00 ft	El. inlet crest		0.00 ft		

***** SITE DATA ***** CULVERT INVERT *****
 INLET STATION (FT) 0.00
 INLET ELEVATION (FT) 654.17
 OUTLET STATION (FT) 212.00
 OUTLET ELEVATION (FT) 654.16
 NUMBER OF BARRELS 2
 SLOPE (V-FT/H-FT) 0.0000
 CULVERT LENGTH ALONG SLOPE (FT) 212.00

***** CULVERT DATA SUMMARY *****
 BARREL SHAPE PIPE ARCH
 BARREL SPAN 5.42 FT
 BARREL RISE 3.33 FT
 BARREL MATERIAL STEEL OR ALUMINUM
 BARREL MANNING'S N 0.024
 INLET TYPE CONVENTIONAL
 INLET EDGE AND WALL PROJECTING
 INLET DEPRESSION NONE

CURRENT DATE: 08-12-1994
CURRENT TIME: 09:10:57

FILE DATE: 08-11-1994
FILE NAME: EX45-1

TAILWATER

***** USER DEFINED CHANNEL CROSS-SECTION
MAIN CHANNEL AND LT & RT OVER BANKS
LEFT CHANNEL BOUNDARY 3
RIGHT CHANNEL BOUNDARY 7
MANNING N LEFT OVER BANK 0.090
MANNING N MAIN CHANNEL 0.080
MANNING N RIGHT OVER BAN 0.090
SLOPE OF CHANNEL (FT/FT) 0.0036

FILE NAME: EX45-1
FILE DATE: 08-11-1994

CROSS-SECTION COORD. NO.	X (FT)	Y (FT)
1	0.00	656.00
2	0.00	655.20
3	30.00	655.00
4	43.00	654.00
5	49.00	653.95
6	53.00	654.00
7	85.00	655.00
8	140.00	656.00

***** UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (CFS)	W.S.E. (FT)	FROUDE NUMBER	DEPTH (FT)	VEL. (FPS)	SHEAR (PSF)
0.00	653.95	0.000	-0.21	0.00	0.00
5.00	654.51	0.164	0.35	0.54	0.08
10.00	654.68	0.170	0.52	0.63	0.10
15.00	654.81	0.175	0.65	0.70	0.11
20.00	654.91	0.178	0.75	0.75	0.12
25.00	655.00	0.180	0.84	0.79	0.13
30.00	655.07	0.183	0.91	0.85	0.15
35.00	655.13	0.186	0.97	0.90	0.16
40.00	655.18	0.188	1.02	0.94	0.17
45.00	655.23	0.190	1.07	0.98	0.19
50.00	655.28	0.192	1.12	1.02	0.20

ROADWAY OVERTOPPING DATA

ROADWAY SURFACE EMBANKMENT TOP WIDTH (FT)	PAVED
48.00	

***** USER DEFINED ROADWAY PROFILE

CROSS-SECTION COORD. NO.	X (FT)	Y (FT)
1	0.00	662.00
2	115.00	661.00
3	190.00	660.60
4	265.00	660.50
5	375.00	660.80
6	500.00	661.00
7	615.00	662.00

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Chapter 5

FLOW IN GUTTERS AND INLETS

Chapter 5 - FLOW IN GUTTERS AND INLETS

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Chapter 5 - FLOW IN GUTTERS AND INLETS

LIST OF PARAMETERS

a	Depth of inlet depression (inches or cm)
A	Drainage area (acres or hectares)
A_g	Clear opening of gutter inlet (ft^2 or m^2)
A_i	Clear opening of grate inlet (ft^2 or m^2)
C	Rational method runoff coefficient
C_w	Weir coefficient for a curb inlet
d	Depth of flow at gutter (ft or m)
d'	Depth of flow in street at some distance from gutter (ft or m)
d_o	Depth of water over orifice (ft or m)
d_w	Depth of water over weir (ft or m)
D_H	Distance from high point to tangent point
D_L	Distance from low point to tangent point
E	Efficiency of curb inlets
E_o	Ratio of flow in depressed section to total gutter flow
g	Acceleration due to gravity (ft/sec^2 or m/sec^2)
G	Tangent grade
h	Curb opening height (height between top of curb opening to bottom of depression) (ft or m)
i	Rainfall intensity (inches/hr)
K	Neenah coefficient for grate capacity
L	Actual curb opening length (ft or m)
L_c	Length of curb opening to intercept 100% of gutter flow (ft or m)
n	Manning's roughness coefficient
P	Perimeter of grate opening (ft or m)
Q_b	Bypassed flow (cfs or m^3/sec)
Q_B	Remaining flow in street between distance x and T (cfs or m^3/sec)
Q_c	Carryover flow from upgrade inlet (cfs or m^3/sec)
Q_G	Gutter flow (cfs or m^3/sec)
Q_i	Capacity of inlet (cfs or m^3/sec)
Q_x	Flow in street between gutter and a distance x from the gutter (cfs or m^3/sec)
S_e	Equivalent cross slope for curb inlets
S_L	Longitudinal slope
S_T	Transverse slope
S_w	Slope of the depressed curb inlet area
T	Width of flow (ft or m)
v	Velocity of flow in gutter (ft/sec or m/sec)
W	Width of depression (ft or m)
W_s	Width of slotted drain inlets (ft or m)
X	Width (ft or m)
y	Depth of flow in street (ft or m)

Chapter 5 - FLOW IN GUTTERS AND INLETS

5.1 - INTRODUCTION

In many drainage systems, stormwater enters the conveyance system through an inlet. Often these openings are located along a gutter which is designed to convey overland flow to the inlets. These inlets may be located directly in the gutter, curb or in both the curb and gutter. It is important that the gutter and inlet be properly designed to adequately collect the stormwater in order to minimize the potential flooding of the roadway, danger to pedestrians, and disruption of traffic.

Figure 5.1.1 is a diagram illustrating the importance of properly designed inlets. In the left-hand side of the figure the encroachment of the stormwater into the street when the inlets are properly spaced is shown. The dashed line is the street encroachment with no carry-over from the upstream inlet and the solid line is encroachment with carry-over. In either case, these flows would not be very likely to disrupt traffic. However, the flow on the right-hand side shows pavement encroachment far into the street for the case with fewer inlets. This would obviously be a hazard to traffic and pedestrians.

In this chapter, the methods for determining the capacity of both gutters and inlets are discussed. Techniques used in the sizing and spacing of various types of inlets are presented. Example problems illustrate the application of these methods. Computer programs which reduce the amount of hand calculations required in designing inlets are provided.

5.2 - FLOW IN GUTTERS

Gutters are used to convey surface runoff into an inlet or other opening so that it may eventually enter the drainage system. The depth of water in the gutter and the top width of the water surface in the street are important design parameters. These values are dependent on the longitudinal slope, S_L , transverse slope, S_T , and roadway Manning's roughness coefficient, n . A relationship between these fixed values, and the unknown depth and the top width can be determined in the following manner.

Flow in a gutter with a curb can be approximated by using a triangular section as shown in Figure 5.2.1. The gutter is on a longitudinal slope of S_L and a transverse slope of S_T .

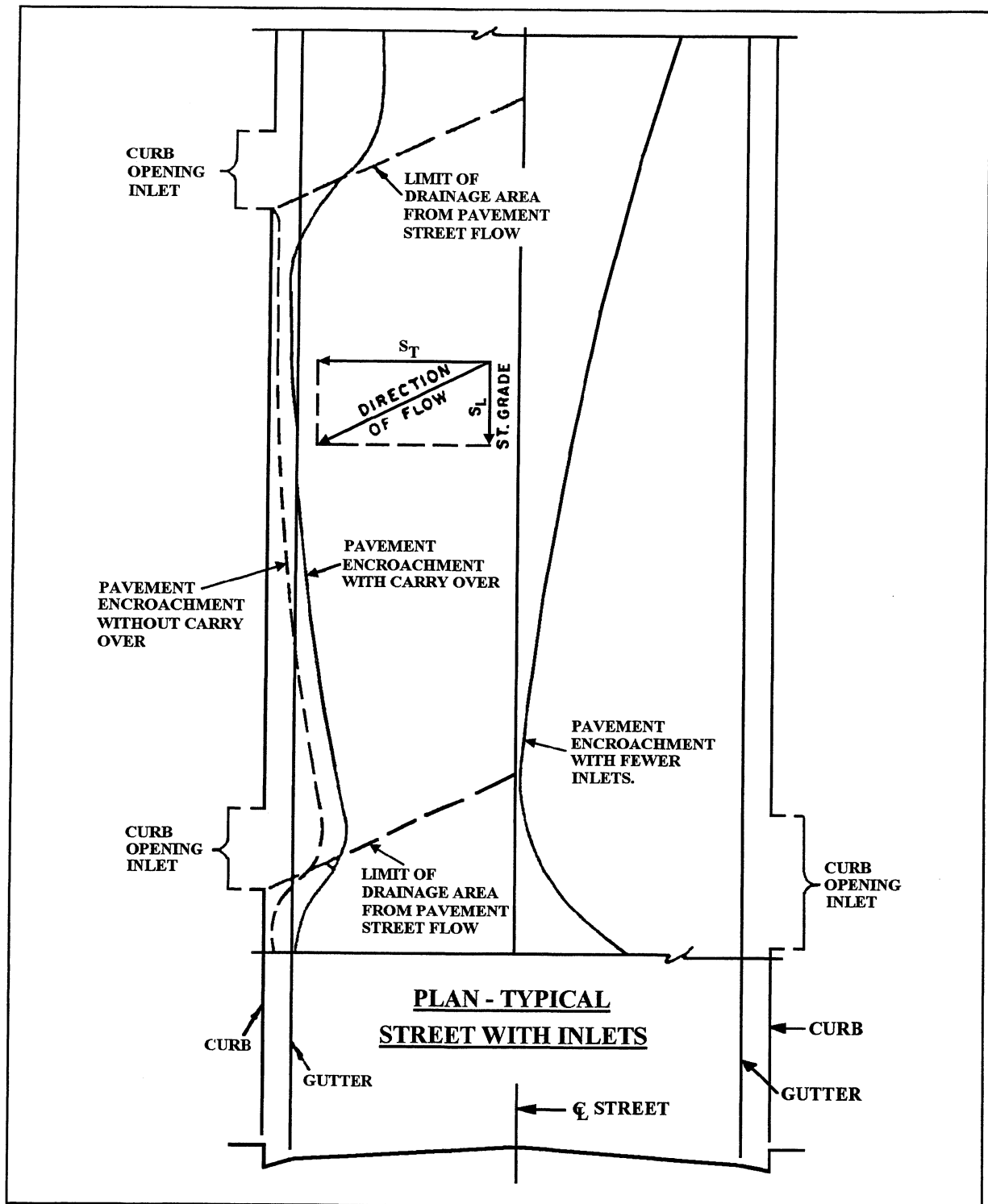


Figure 5.1.1
Diagram of Gutter and Pavement Flow Patterns (Wright, 1968)

The incremental flow rate, dQ_G , in the gutter is:

$$dQ_G = v y \, dx \quad (5.2.1)$$

where v is the velocity in the cross section of width dx (ft/sec), dx is the incremental width of cross section (ft), and y is the depth of flow at dx (ft).

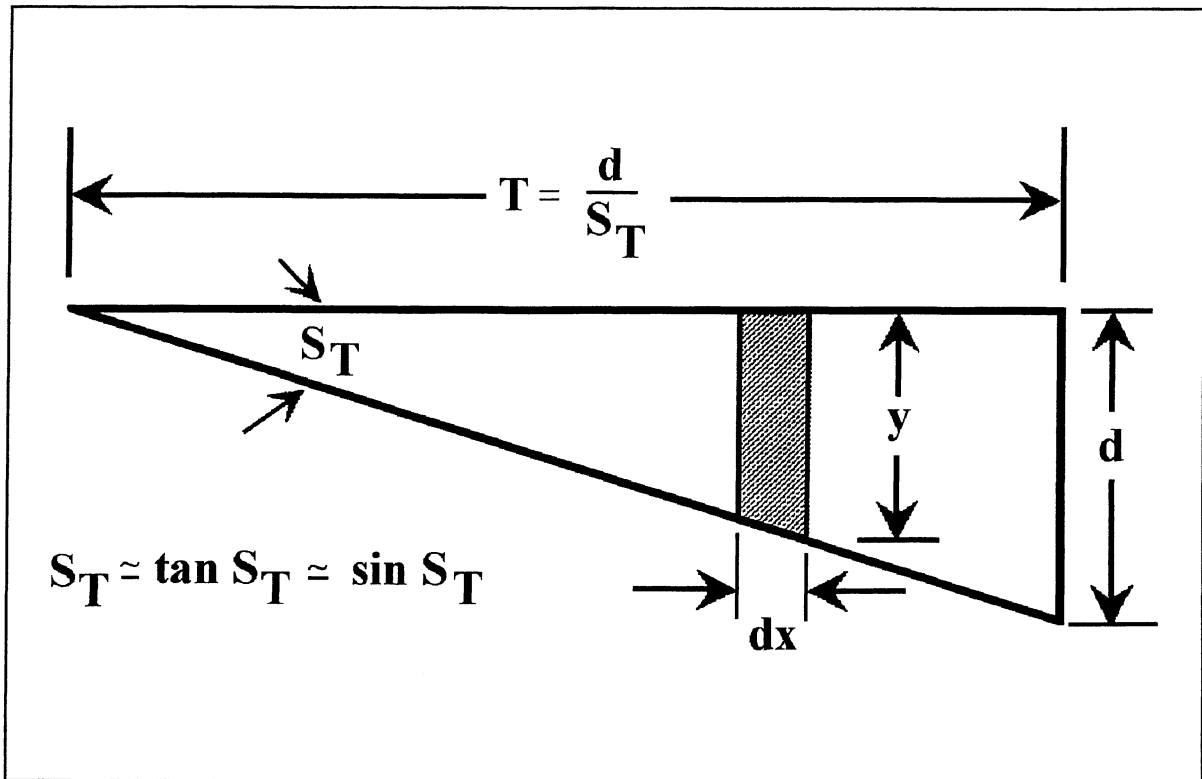


Figure 5.2.1 Flow in a Gutter

The velocity, v , may be estimated using Manning's Equation (Equation 4.2.1) with slope, S_L , and the hydraulic radius equal to the depth, y ,

$$v = \frac{1.49}{n} y^{2/3} S_L^{1/2} \quad (5.2.2)$$

where n is Manning's roughness coefficient (Table 4.2.1).

Substituting this relationship into Equation 5.2.1, the following equation is obtained.

$$dQ_G = \left[\frac{1.49}{n} y^{5/3} S_L^{1/2} \right] dx \quad (5.2.3)$$

The incremental width, dx , may be expressed as dy/S_T , where S_T is the transverse slope of the cross-section (because $dy/dx = \tan S_T = S_T$).

Substituting this into Equation 5.2.3 and integrating y from 0 (zero) to d gives:

$$Q_G = \frac{1.49}{n} \frac{S_L^{1/2}}{S_T} \frac{3}{8} d^{8/3} = \frac{0.56}{n} \frac{S_L^{1/2}}{S_T} d^{8/3} \quad (5.2.4)$$

The depth of flow against the gutter is given by Equation 5.2.5, while the width of the flow, T , is determined from:

$$T = \frac{d}{S_T}$$

$$d = \left[1.79 Q_G n \frac{S_T}{S_L^{1/2}} \right]^{3/8} \quad (5.2.5)$$

Figure 5.2.2 is a nomograph solution of Equation 5.2.4 (to find Q_G). It may also be used to calculate the depth of flow, d , (Equation 5.2.5).

After calculating the runoff from a subbasin and knowing the roadway slopes, S_T and S_L , and pavement roughness n , the depth and width of flow can be calculated. When the depth reaches a height greater than that of the curb or the width of flow extends too far into the street, an inlet must be placed to capture the flow.

Sometimes it is desired to know the discharge and depth in only a part of the channel. This information is often used in determining the capacity of grate inlets. Figure 5.2.3 shows a gutter cross-section. It is desired to find the discharge only in the portion of the channel having width x . The total discharge, Q_G , in the cross-section may be found using Equation 5.2.4 or Figure 5.2.2. The depth of flow at x can be calculated using Equation 5.2.6.

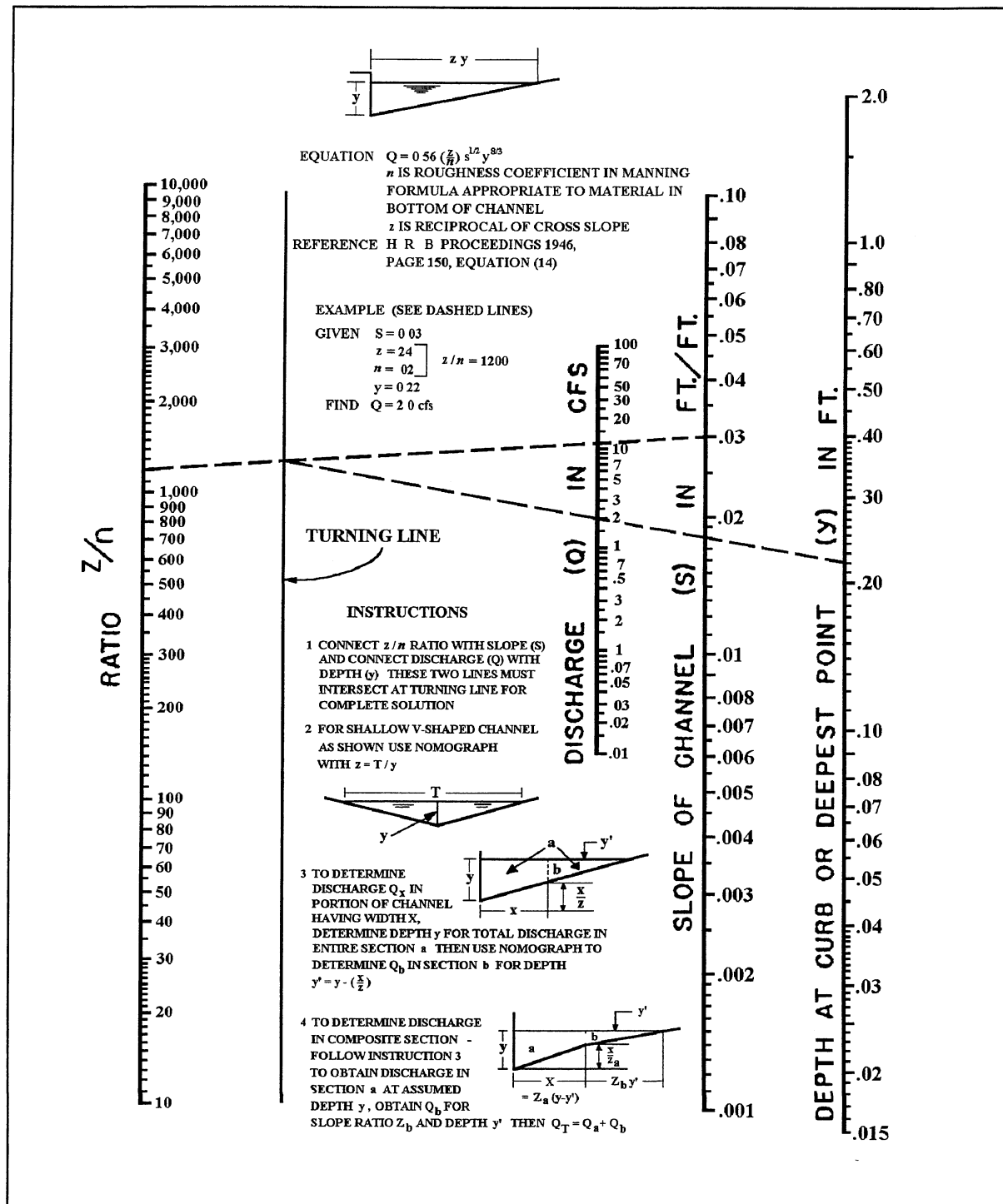


Figure 5.2.2
 Nomograph Solution of Equations 5.2.4 and 5.2.5
 (Bureau of Public Roads, 1968)

$$d' = d - x S_T \quad (5.2.6)$$

The value of d' can then be used with Equation 5.4 or Figure 5.3 to calculate the flowrate, Q_B . Then the discharge in the portion of the channel within a distance x from the gutter, Q_X , can be calculated as:

$$Q_X = Q_G - Q_B \quad (5.2.7)$$

The information presented here is required to design inlets properly. Applications of these relationships are illustrated in Example 5.2.1.

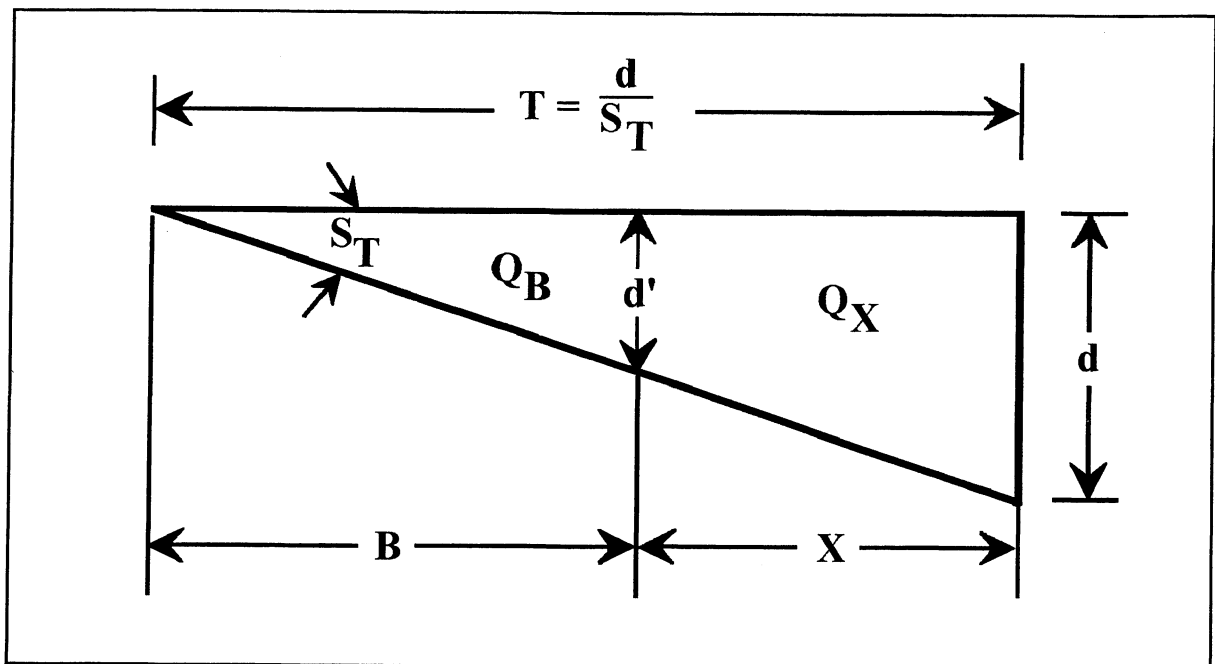


Figure 5.2.3 Gutter Cross-section

Example 5.2.1 (Flow in gutters)

Given: a concrete gutter with a rough asphalt pavement and a flowrate of 3.0 cfs, transverse slope of 1/4 inch per foot, and longitudinal slope of 3%.

Calculate: the depth of flow, the top width and the flowrate within 2 feet of the curb and depth at a distance of 2 feet from the curb.

From Table 4.2.1, the roughness is found to be 0.015. The transverse slope is calculated as:

$$S_T = \frac{1/4}{12} = \frac{1}{48} = 0.02$$

The depth of water at the gutter can be calculated using Equation 5.2.5.

$$d = \left[1.79 Q_G n \frac{S_T}{S_L^{1/2}} \right]^{3/8} = \left[(1.79) (3.0) (0.015) \frac{(0.02)}{(0.03)^{1/2}} \right]^{3/8} = 0.17 \text{ ft.}$$

Therefore the top width is:

$$T = \frac{d}{S_T} = \frac{0.17}{0.02 \text{ ft/ft}} = 8.50 \text{ ft.}$$

The depth of flow two feet from the gutter is calculated using Equation 5.2.6.

$$d' = d - x S_T = 0.17 - 2 (0.02) = 0.13 \text{ ft.}$$

The flowrate in the street from 2 to 8.50 feet (Q_B) is computed using Equation 5.2.4.

$$Q_B = \frac{0.56}{n} \frac{S_L^{1/2}}{S_T} d'^{8/3} = \frac{0.56}{0.015} \frac{(0.03)^{1/2}}{(0.02)} (0.13)^{8/3} = 1.40 \text{ cfs}$$

The flowrate in the street between the gutter and two feet from the gutter (Q_x) is then computed as:

$$Q_x = Q_G - Q_B = 3 - 1.40 = 1.60 \text{ cfs}$$

5.3 - FLOW IN INLETS

There are basically four types of inlets which are used in drainage design: gutter inlets; combination inlets; curb opening inlets; and slotted drain inlets. The capacity of each of these depends upon the inlet size, longitudinal and transverse slopes, street roughness, and the depth that the inlet is depressed (for gutter and combined inlets). The methods used in determining the capacity of the inlets as a function of the previously mentioned variables are discussed in this section.

Regardless of inlet type, there are some general design considerations. A few of these are listed below:

1. Is the inlet to be placed on a continuous grade or in a "sag"?
2. Is the inlet to be designed for 100% efficiency? (Efficiency is the ratio of the runoff passing over the inlet to the runoff captured).
3. Is the inlet to be depressed? (For gutter and combined inlets only).
4. Will the inlet be placed in an area where it will be susceptible to clogging?

These questions will help the designer to select the inlet(s) which will best suit the design criteria. It should be pointed out that regardless of the storm sewer design capacity, the inlet must be able to admit the stormwater into the storm system or flooding will occur with a frequency greater than the sewer system design would indicate.

5.3.1 - Gutter Inlets

A gutter inlet is an opening in the gutter through which water is admitted into the storm sewer system. The capacity of a gutter inlet decreases with an increase in longitudinal slope, and increases with an increase in transverse slope, grate length, width depression (described below) and grate efficiency. It has also been shown (Larson, 1948) that the capacity of a gutter inlet is increased by allowing a small percentage of the flow to bypass the grate (possible only on a continuous grade). This is achieved by the increased depth, and therefore an increase in the quantity of water captured. In order to intercept all the water, the grate length must be increased. As an example, an inlet grate with a constant transverse slope, S_T , may intercept 82% of the flow when the grate width is 50% of the total flow width. However, to capture the remaining 18% of the flow, the grate length must be doubled. This is shown qualitatively in Figure 5.3.1 where the grate length, L , would have to be extended to L_1 to capture all of the flow Q_G .

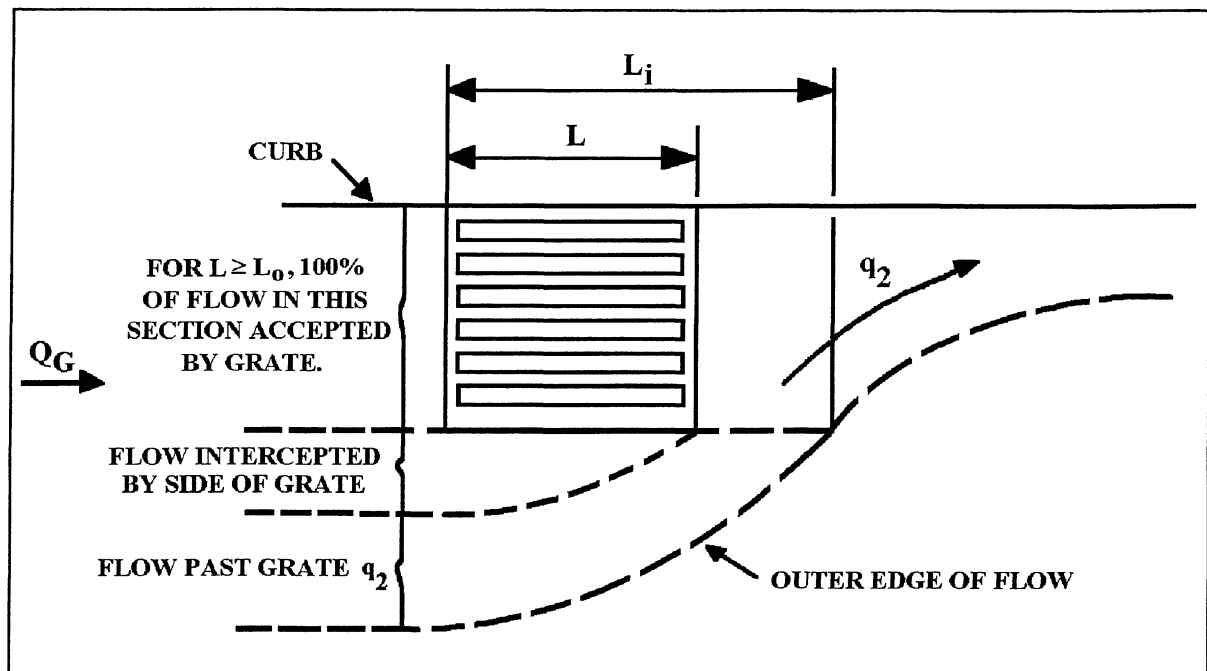


Figure 5.3.1
Plan of Grated Inlet showing Flow Lines (Wright, 1968)

The inlet grate must not be selected only on the basis of capacity. The potential for flooding caused by clogging of the grate with debris may dictate the use of curb inlets or combination inlets. Also, longitudinal parallel bars may cause a hazard to bicyclists and depressions may be dangerous in high speed traffic.

There are many possible bar configurations for the gutter grates. Various companies have published grate size data for an assortment of applications. (Neenah, East Jordan Iron Works, etc.) See References at the end of this Chapter.

For inlets which are on a continuous grade, the capacity of a grate inlet may be easily calculated with a method developed by Neenah (1987). The discharge-depth relationship for a grate inlet is written as

$$Q_i = K d^{5/3} \quad (5.3.1)$$

where Q_i is the capacity of grate inlet (cfs), K is the coefficient dependent on the longitudinal slopes, S_L , transverse slopes, S_T , and grate configuration, and d is the depth of flow (ft).

Graphs published by Neenah (Neenah, 1987) for many of their manufactured grates present K as a function of S_T and S_L . Figure 5.3.2 is a typical chart. Each of these graphs published by Neenah is the result of 96 separate test points. The charts are applicable for Manning's roughness coefficient, n , between 0.013 - 0.014, and longitudinal slopes between 1 and 6 percent.

The maximum allowable depth, d , in Equation 5.3.1 may be obtained from a rating table or Equation 5.2.5. The spacing between the inlets will depend on the maximum allowable depth in the gutter or the maximum width of flow in the street. Example 5.3.1 illustrates the use of the charts. (NOTE: The use of Neenah Charts is not to be interpreted as an endorsement of their products by the author or HERPICC).

For gutter inlets located in a "valley", a sump condition will exist and all the water will be captured eventually. The rate at which the flow enters the inlet is determined by the depth of water above the grate. For depths less than 0.3 feet, the inlet acts as a weir and the discharge is calculated from Equation 5.3.2,

$$Q_i = 3.0 P(d_w)^{1.5} \quad (d_w < 0.3 \text{ ft}) \quad (5.3.2)$$

where Q_i is the discharge rate into the inlet (cfs), d_w is the depth of water above grate top for weir flow (ft) and P is the perimeter of grate opening (neglect bars and side against curb) (ft).

For depths exceeding 0.4 feet, the grate starts to act as an orifice. For orifice condition the discharge is calculated by Equation 5.3.3.

$$Q_i = 4.89 A_i (d_o)^{0.5} \quad (d_o > 0.4 \text{ ft}) \quad (5.3.3)$$

where A_i is the clear opening of grate (ft^2) (obtained from grate manufacturer catalogs) and d_o is the depth of water above the grate top for orifice flow (ft).

The U.S. Dept. of Transportation (1984) recommends that a factor of safety of two be used when the danger of the grate clogging exists. The use of these equations is illustrated by Example 5.3.2.

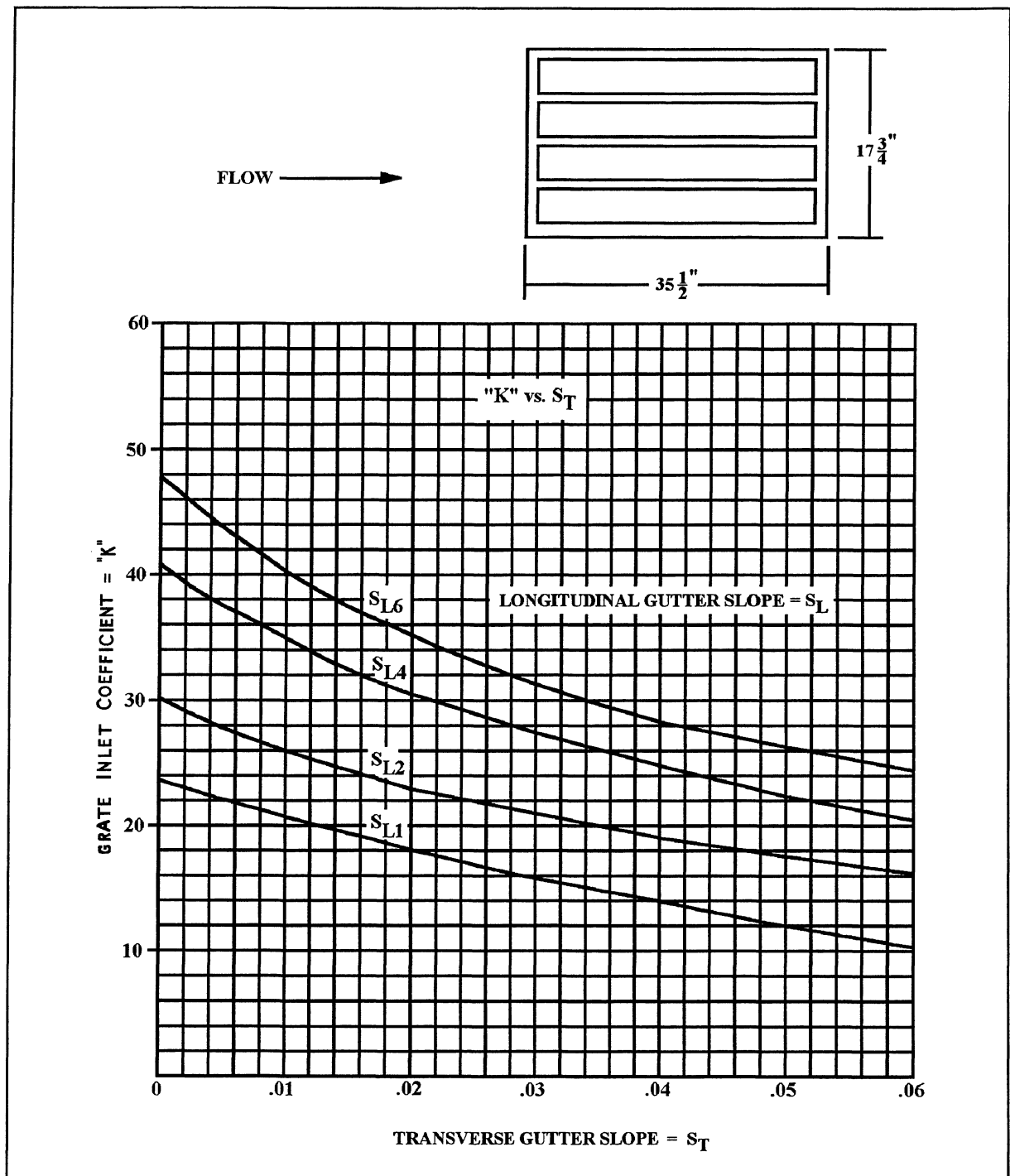


Figure 5.3.2
 Typical Inlet Grate Capacity Chart (Neenah, 1987)
 Note: S_{L6} = 6%, etc.

Example 5.3.1 (Gutter inlets - continuous grade)

Given: the grate shown in Figure 5.3.2 with a longitudinal slope of 4 %, transverse slope of 2 %, gutter flow rate of 3.0 cfs and roughness coefficient of 0.014.

Calculate: the grate capacity.

From Figure 5.3.2, K is found to be 30.3. The depth is found using Equations 5.2.5.

$$d = \left[1.79 (3.0) (0.014) \frac{0.02}{(0.04)^{1/2}} \right]^{3/8} = 0.16 \text{ ft.}$$

Therefore, by using Equation 5.3.1 the grate capacity is found as:

$$Q_1 = K d^{5/3} = 30.3 (0.16)^{5/3} = 1.43 \text{ cfs}$$

The inlet efficiency is therefore $1.43/3.0 = 48\%$. It would be necessary to consult the catalog and select a different grate or use two grates in series if a greater interception efficiency is needed.

Example 5.3.2 (Gutter inlets - sump condition)

Given: a gutter inlet, located in a sag, with a width of 2.5 feet, length of 2.0 feet, and a 50% clear opening.

Calculate: the depth of flow over the inlet when the discharge into the grate is 1.0 cfs and 10.0 cfs.

For a flow of 1 cfs, start by using Equation 5.3.2.

$$Q_1 = 3.0 P (d_w)^{1.5} \quad \therefore \quad d_w = \left[\frac{Q_1}{3.0 P} \right]^{0.67}$$

$$P = 2.0 + 2.5 + 2.5 = 7.0 \text{ ft.}$$

Using the recommended factor of safety of 2, $P = 3.5$. Therefore, for a flow of 1.0 cfs (using Equation 5.3.2 and rearranging to find d_w (the depth of flow for weir flow):

$$d_w = \left[\frac{1.0}{(3.0)(3.5)} \right]^{0.67} = 0.21 \text{ ft.} < 0.3 \text{ ft.} \quad (\text{weir flow})$$

Check the depth for orifice flow (d_o):

$$d_o = \left[\frac{1.0}{(4.89)(1.25)} \right]^2 = 0.027 \text{ ft.}$$

Because the depth is less than 0.3 feet for both d_w and d_o , Equation 5.3.2 is applicable for $Q_i = 1.0$ cfs. For $Q_i = 10.0$ cfs, the depth is calculated using Equation 5.3.3.

$$Q_i = 4.89 A d_o^{0.5} \quad \dots \quad d_o = \left[\frac{Q_i}{4.89 A} \right]^2 \quad (\text{orifice flow})$$

where $A = (2.5)(2.0)(0.5) = 2.50 \text{ ft}^2$ or using the factor of safety, $A = 1.25 \text{ ft}^2$. Therefore,

$$d_o = \left[\frac{10.0}{(4.89)(1.25)} \right]^2 = 2.68 \text{ ft.} \geq 0.4 \text{ ft.} \quad (\text{orifice flow})$$

$$d_w = \left[\frac{10.0}{(3.0)(3.5)} \right]^{0.67} = 0.96 \text{ ft.} \quad (\text{weir flow})$$

Checking for both weir and orifice flow, the results indicate that the inlet acts as an orifice because the depth is greater than 0.4 feet. The resulting depth for $Q_i = 10.0$ cfs is 2.68 feet.

5.3.2 - Curb Inlets

A curb inlet, as the name implies, is located directly in the curb, causing little interference to traffic. This type of inlet is not as susceptible to clogging and may be used for streets with mild slopes. The capacity of a curb inlet depends on street slopes (longitudinal and transverse), curb opening length, the top width and depth of the flow at the curb, and the depth and width of the inlet depression. The inlet depression and geometry has a significant effect on its capacity. The symbols and notation for curb inlets are shown on Figure 5.3.3.

The U.S. Dept. of Transportation has revised the methodology for determining the curb inlet lengths required for a set of inputs. The total interception of the gutter flow on a section of pavement with a uniform transverse slope can be calculated by Equation 5.3.4,

$$L_c = 0.6 Q_G^{0.42} S_L^{0.3} \left(\frac{1}{n S_T} \right)^{0.6} \quad (5.3.4)$$

where Q_G is the total gutter flow (cfs), and L_c is the length of curb opening required to intercept 100% of the gutter flow (feet).

The efficiency of curb inlets shorter than L_c is determined by Equation 5.3.5,

$$E = 1 - \left(1 - \frac{L}{L_c} \right)^{1.8} \quad (5.3.5)$$

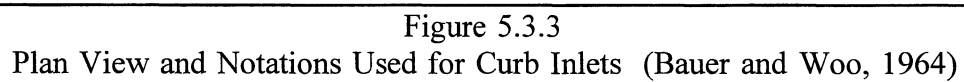
where L is the actual curb opening length (feet).

The length of opening for depressed inlets is computed by Equation 5.3.4 but substituting S_e in lieu of S_T . This is the equivalent cross-slope which is calculated by Equation 5.3.6, and shown on Figure 5.3.3,

$$S_e = S_T + S_w E_o \quad (5.3.6)$$

where S_w is the transverse slope of the gutter [$S_w = (a/12W)$], a is the depth of inlet depression (inches), W is the width of depression (feet), and E_o is the ratio of flow in depressed section to total gutter flow.

A graphical solution to E_o is provided by Figure 5.3.4. The use of these equations is illustrated by Example Problem 5.3.3.



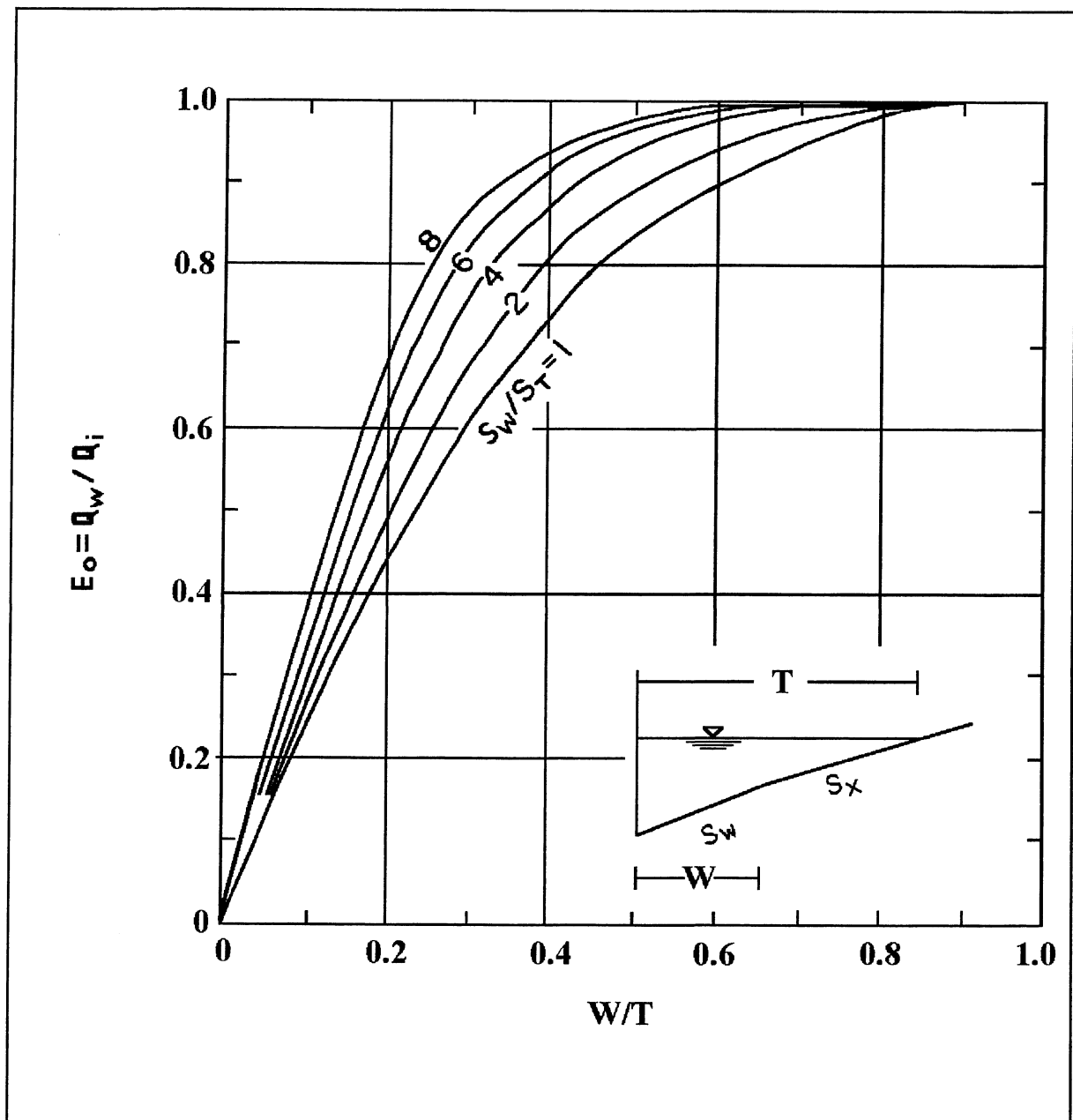


Figure 5.3.4
Ratio of Frontal Flow to Total Gutter Flow (U.S. DOT, 1969)

For a sump condition, the capacity of the inlet depends upon the depth of water at the curb, the opening length and the height of the curb opening. The inlet will act as a weir until the entrance is submerged. When the depth is greater than 1.4 times the curb opening height, the inlet will operate as an orifice. The flow is indeterminate for the intermediate region.

The weir coefficient for a curb inlet is less than the usual coefficient because experimental measurements for determining the coefficients were taken in the curb and not in the inlet, and because there is a drawdown in the water surface between the point where the measurements are made and the weir. The weir location for a depressed curb inlet is at the edge of the gutter while the weir length is dependent on the width of the depression and the curb opening length. For a non-depressed curb inlet, the weir location is at the lip of the opening and the corresponding length is equal to the inlet length. The weir coefficient for depressed curb inlets and those without depression are approximately equal. For a depressed curb inlet which acts as a weir, the equation for the interception capacity is given by Equation 5.3.7,

$$Q_1 = C_w (L + 1.8W) d^{1.5} \quad \left(d \leq h + \frac{a}{12} \right) \quad (5.3.7)$$

where C_w is equal to 2.3, W is the width of depression (feet), a is the depth of depression (inches), and h is the curb opening height (see Figure 5.3.3)

For a curb inlet without depression which acts as a weir ($W=0$), the equation for the interception capacity is Equation 5.3.8.

$$Q_1 = C_w L d^{1.5} \quad (d \leq h) \quad (5.3.8)$$

A depressed curb inlet acts as an orifice when the depth exceeds 1.4h. The interception equation for this condition is Equation 5.3.9,

$$Q_1 = 0.67 A_g \left[2g \left(d - \frac{h}{2} \right) \right]^{1/2} \quad (d \geq 1.4 h) \quad (5.3.9)$$

where A_g is the area of opening equal to hL .

Equation 5.3.9 is also applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter depression.

Example 5.3.3 (Curb inlet - continuous grade)

Given: a gutter with a flowrate of 2.0 cfs, transverse slope of 2 percent, longitudinal slope of 3 percent, depression width of 2 feet, and inlet depression of 2 inches. The gutter has a concrete broom finish ($n = 0.016$).

Calculate: the intercepted flow for curb inlet lengths of 5, 10 and 15 feet.

From Equation 5.2.5 the depth of flow in the channel at the curb is found to be:

$$d = [1.79 (2.0) (0.016) \frac{(0.02)}{\sqrt{0.03}}]^{3/8} = 0.15 \text{ ft.}$$

The gutter flow spread is:

$$T = \frac{d}{S_T} = \frac{0.15}{0.02} = 7.5 \text{ ft.}$$

The slope of the depressed area is:

$$S_w = \frac{a}{12W} = \frac{2}{24} = 0.083$$

$$\text{The ratio of } \frac{S_w}{S_T} = \frac{0.083}{0.02} = 4.17$$

$$\text{The ratio of } \frac{W}{T} = \frac{2}{7.5} = 0.27$$

From Figure 5.3.4, the ratio $E_o = 0.73$

Therefore, S_e is calculated from Equation 5.3.6:

$$S_e = S_T + S_w E_o = 0.02 + 0.083 (0.73) = 0.081$$

Example 5.3.3 (Continued)

The curb opening length for 100% efficiency can now be calculated from Equation 5.3.4 as:

$$L_C = 0.6 Q^{0.42} S_L^{0.3} \left[\frac{1}{n S_e} \right]^{0.6} = (0.6) (2^{0.42}) (0.03^{0.3}) \left[\frac{1}{(0.016)(0.081)} \right]^{0.6}$$

$$L_C = 15.14 \text{ ft.}$$

The efficiency for a five foot curb opening can be calculated from Equation 5.3.5 as:

$$E = 1 - \left(1 - \frac{L}{L_C} \right)^{1.8} = 1 - \left(1 - \frac{5}{15.14} \right)^{1.8} = 0.51$$

This means that the total intercepted flow is (0.51)(2 cfs) or 1.02 cfs. The same procedure is used to calculate the efficiency and flows captured by a 10 and 15 foot long opening. The results are summarized below:

<i>Length (ft)</i>	<i>E_o</i>	<i>Q_i (cfs)</i>
5	0.51	1.02
10	0.86	1.72
15	1.00	2.00

Example 5.3.4 (Curb inlet - sump condition)

Given: a curb inlet located in a sump with the following information.

$$\begin{array}{ll} S_T = 0.05 & L = 5 \text{ feet} \\ h = 5'' = 0.42' & T = 8 \text{ feet} \end{array}$$

Calculate: the intercepted flow for an *undepressed* curb opening.

$$d = T S_T = (8) (0.05) = 0.4 \text{ ft.} = 4.8 \text{ inches}$$

Since $d < h$, the flow is weir type and Q_i is computed from Equation 5.3.8.

$$Q_i = C_w L d^{1.5} = (2.3) (5.0) (0.4)^{1.5} = 2.91 \text{ cfs}$$

For a *depressed* curb opening with $a = 2''$, $W = 2$ feet (all other information remaining the same).

$$h + \frac{a}{12} = 0.42 + \frac{2}{12} = 0.59 \text{ ft.} > (1.4) d \quad \therefore \text{ use Equation 5.3.7}$$

$$Q_i = 2.3 (L + 1.8W) d^{1.5} = 2.3 (5.0 + 1.8(2)) (0.4)^{1.5} = 5.0 \text{ cfs}$$

The depressed curb opening inlet is about 72% more efficient than the undepressed opening.

5.3.4 - Combined Inlets

Combined inlets have both a gutter and curb inlet and are used when there is a possibility that the gutter grate may be clogged frequently. The capacity of a combined inlet on a continuous grade may be computed by ignoring the curb inlet and calculating the interception of the gutter inlet alone.

Combination inlets are strongly recommended when a sump condition exists. The curb inlet will provide relief if the gutter grate becomes clogged. The capacity of the grate may be computed using Equation 5.3.2 or Equation 5.3.2 depending upon the depth of the water. The safety factor of two, however, is usually ignored.

5.3.5 - Slotted Drain Inlets

A typical slotted inlet drain is shown on Figure 5.3.5. This type of inlet consists of a long narrow inlet acting as a slot into a pipe. These inlets are used on curbed or uncurbed sections and minimize traffic interference. The disadvantages of the slotted drain inlet are the deposition in the pipe and the fact that there is only a minimal amount of information available on their ability to handle debris. However it is easy and relatively inexpensive to add length to a slotted drain inlet. The U.S. Dept. of Transportation analysis of the slotted drain inlets suggests that when the width of the drain is greater than 1.75 inches, the interception capacity be computed by using the same methods as a curb inlet. In other words, Equations 5.3.4 and 5.3.5 are used to compute the capacity of a slotted inlet on grade.

Slotted drains located in a sag act as weirs until a depth of about 0.2 feet, depending on the slot width and length. When the depths exceed 0.4 feet slotted drains act as orifices. Depths between 0.2 feet and 0.4 feet are in a transition stage. When the slot acts as an orifice, the intercepted flow is calculated by Equation 5.3.10,

$$Q_i = 0.8 L W_s (2gd)^{0.5} \quad (d \geq 0.4 \text{ ft}) \quad (5.3.10)$$

where g is the acceleration due to gravity and W_s is the width of slot (feet); for $W_s = 1.75$ ", $Q_i = 0.94 L d^{0.5}$.

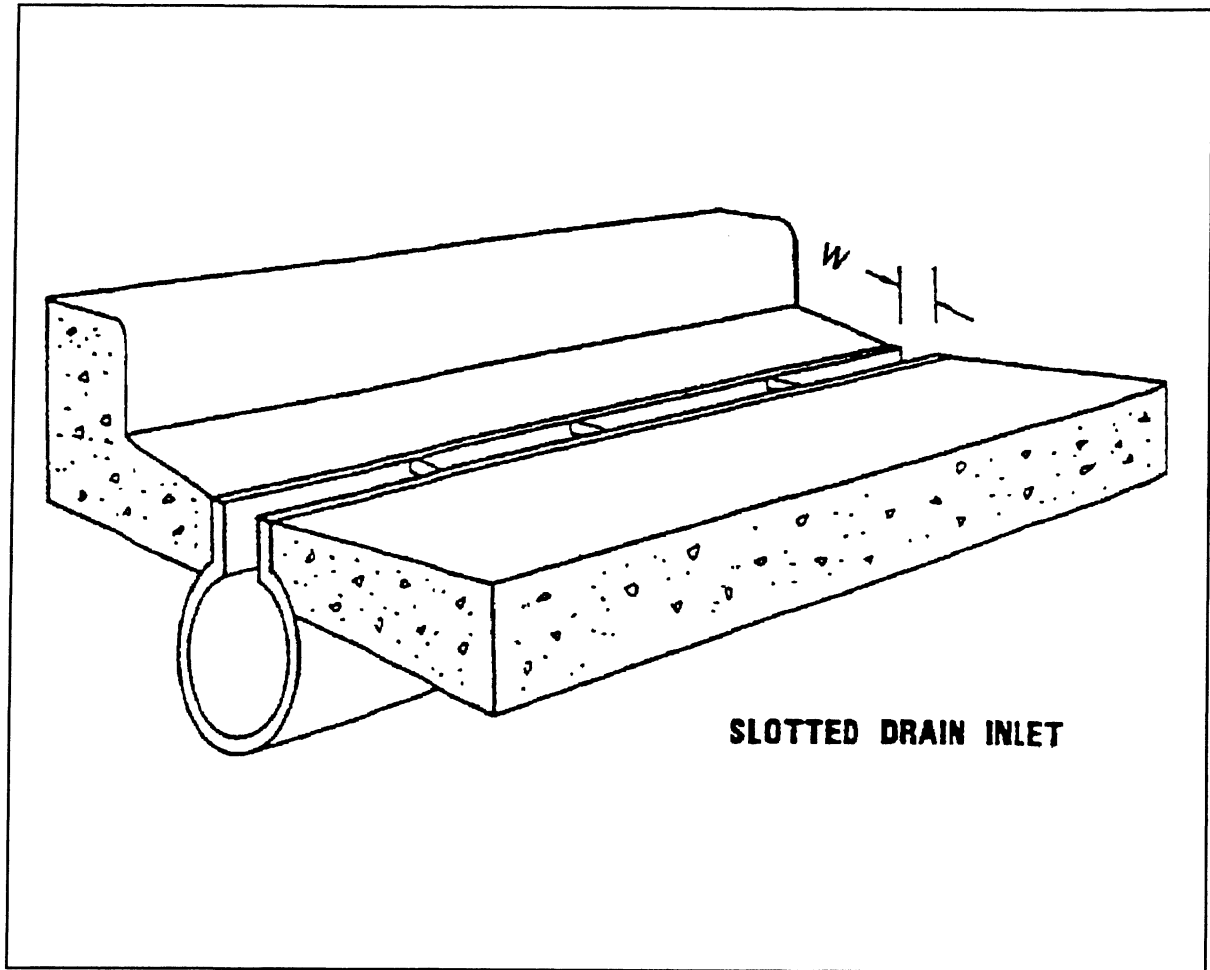


Figure 5.3.5
Typical Slotted Drain Inlet (U.S. DOT, 1984)

For depths between 0.2 and 0.4 feet the capacity can be calculated using the orifice equation with the coefficient dependent on depth, slot width, and length. A graphical solution of a slotted drain inlet in a sump condition is presented in Figure 5.3.6.

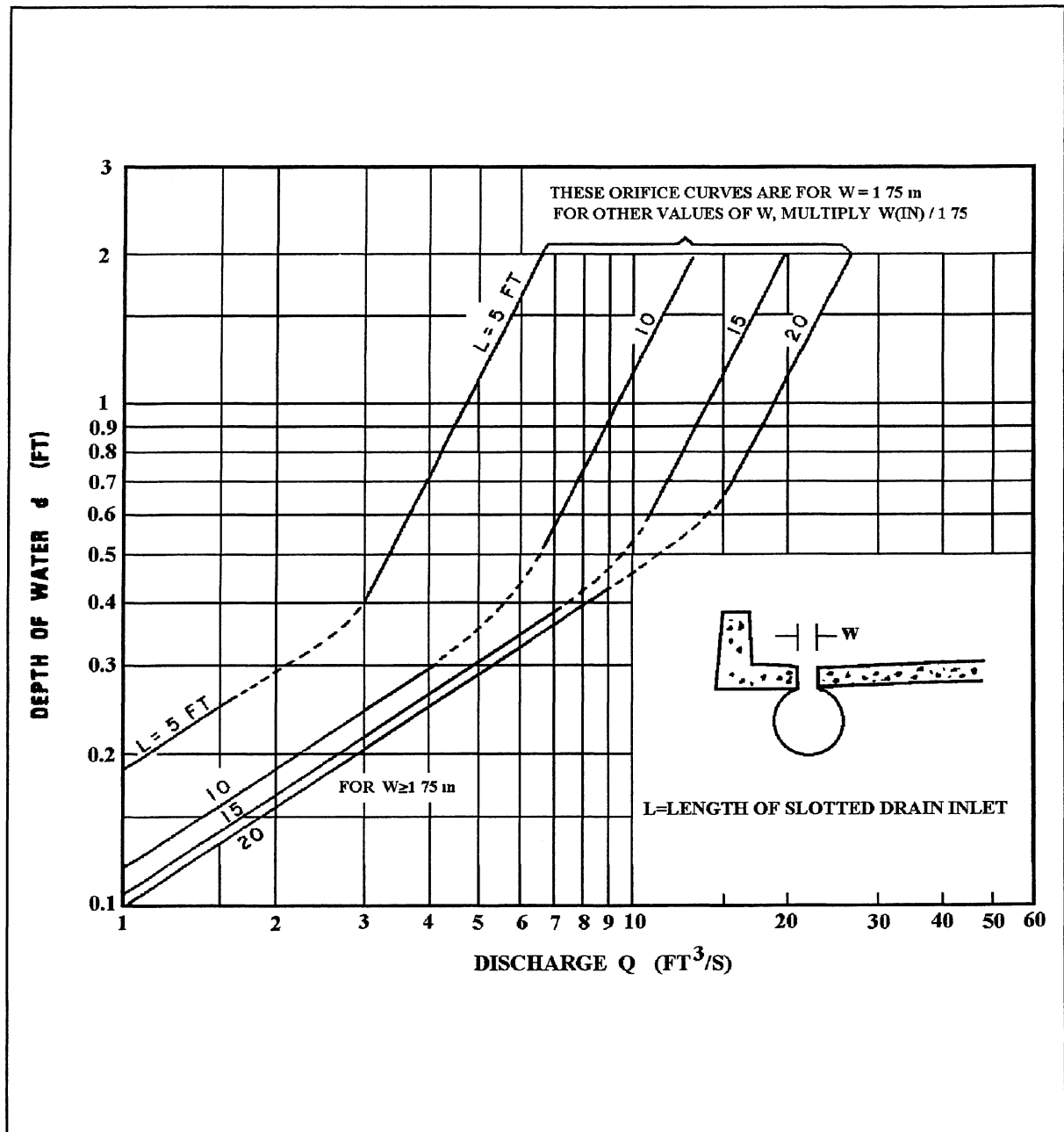


Figure 5.3.6
Slotted Drain Inlet Capacity in Sump Locations, (U.S. DOT, 1984)

Example 5.3.5 (Slotted Drain Inlet - Sump Condition)

Given: a slotted inlet with the following information:

$$\begin{array}{ll} W_s = 1.75 \text{ in.} & S_T = 0.05 \\ Q = 5 \text{ cfs} & S_L = 0.02 \end{array}$$

Calculate: the length of slotted drain inlet to limit depth at curb to 0.3 feet and 0.6 feet.

- a. For $d = 0.30$, we have transition flow, and from Figure 5.3.6 the length of inlet required to intercept 100% of the flow is determined to be 15 feet.
- b. For $d = 0.60$ feet, we have orifice flow and we can either use Figure 5.3.6 or Equation 5.3.10.

$$Q_i = 0.94LD^{0.5}, \quad \text{for } W_s = 1.75 \quad ..$$

$$L = \frac{Q_i}{(0.94)(d)^{0.5}} = \frac{5 \text{ (cfs)}}{(0.94)(0.6)^{0.5}} = 6.9 \approx 7 \text{ ft.}$$

5.4 - INLET DESIGN

The design criteria and methods for the design of stormwater inlets are presented in this section. This information will aid in inlet selection and spacing. Listed below are some general requirements which must be considered prior to inlet selection:

1. The inlet should have sufficient capacity to intercept stormwater from the gutter.
2. The inlet should be designed so as to control the ponding of flow.
3. The inlet should be able to pass small debris (e.g. leaves) while screening out larger, harmful debris (e.g. branches of trees).
4. The inlet should have sufficient strength to resist traffic loadings.
5. Inlet grates should not be hazardous to bicycles.

5.4.1 - Design Criteria

The following is a list of some general criteria for the design of stormwater inlets:

Return Period: The inlet should be designed for a ten (10) year return period. (This is a requirement of the Indiana Department of Transportation, Hydraulics Engineering Section).

Encroachment: The spread, top width or encroachment of stormwater into the street should be less than 4 feet for streets without parking; and less than 12 feet or a maximum depth of 0.35 feet for a street with parking.

Minimum Gutter Slope: The minimum longitudinal slope of the gutter, S_L , is 0.4%,

Minimum Transverse Slope: The minimum transverse slope of the street, S_T , is 1%.

Inlet Locations: Inlets should be placed at all low points in the grade of the gutter and at intersections to prevent stormwater from flowing across traffic lanes and crosswalks. In addition, inlets should be placed when the top width (see above) or the flow depth exceeds the maximum allowable limits.

Spacing of Inlets: Inlets should be spaced so as to capture the flow from the drainage area contributing to the inlet plus any stormwater not intercepted by the stream inlet (carryover) less any flow intentionally bypassed. Using the rational method to calculate the flow from the contributing drainage area (see Section 3.2) the flow in the gutter can be written as,

$$Q_G = CiA + Q_c \quad (5.4.1)$$

where Q_G is the flowrate in gutter at the inlet location (cfs), Q_c is the carryover from upgrade inlet (cfs), (Q_c is zero for the first inlet), C is the runoff coefficient, A is the drainage area (acres), and i is the rainfall intensity (inches/hr).

The capacity of the inlet Q_i which is a function of the longitudinal and transverse slopes and inlet geometry can be determined by using the appropriate equations or figures. The depth of flow used in these equations or figures is determined by using the criteria presented above under "Encroachment". If the spacing is to be the maximum distance apart, the inlet capacity is found by Equation 5.4.2,

$$Q_i = Q_G - Q_b \quad (5.4.2)$$

where Q_b is the flow bypassed by this inlet (cfs). (This could be zero for 100% efficiency)

The runoff coefficient C for the contributing area is calculated and it is recommended (Izzard, 1946) that a five-minute time of concentration be used to compute the rainfall intensity. This is usually a conservative assumption but it is also the maximum intensity which is usually found on an intensity-duration-frequency curve, for a given frequency.

The contributing area is assumed to be the product of the length and a constant width. If this assumption and Equation 5.4.2 are substituted into Equation 5.4.1, the length between inlets is given by Equation 5.4.3,

$$L = \frac{43,560 (Q_i + Q_b - Q_c)}{Ciw} \quad (5.4.3)$$

where L is the distance between inlets (ft), w is the width of contributing area (ft) (assumed to be a constant), C is the composite runoff coefficient for contributing area (see Chapter 3). NOTE: 43,560 converts acres to square feet and $Q_i + Q_b = Q_G$.

Figure 5.4.1 is included to aid in the computations for a stormwater collection system. This method is illustrated in Example 5.4.1.

[illegible]

Figure 5.4.1
Computation Sheet for Inlet Spacing

Table 5.4.1 shows the suggested inlet spacing for the allowable water spread. Main trunk lines should be sized with pipe flowing full, but not placed under pressure for the 10-year peak discharge (Q_{10}). The hydraulic gradeline shall not exceed the top of the inlet or manhole for the 50-year peak discharge (Q_{50}). (INDOT, 1993).

Table 5.4.1
Allowable Water Spread (INDOT, 1993)

TYPE OF FACILITY	ALLOWABLE SPREAD "T"
Freeways	Edge of travel lane for Q_{50}
Multilane Non-Freeways	Spread acceptable across one entire travel lane for Q_{10}
All Two-Lane Freeways	4 feet onto travel lane for Q_{10}
Bridge Decks	4 feet onto travel lane for Q_{10}
Ramps	8 feet of roadway must remain clear of water during Q_{10}

It is often necessary to locate an inlet along a vertical curve or on a roadway with multiple slopes. With this situation, the equivalent longitudinal slope is a function of the vertical curve grade and the distance from the high or low point to the tangent point. The terms used in determining S_L for a vertical curve are presented in Figure 5.4.2.

For an inlet located on the vertical crest curve, the mean slope, \overline{S}_L , is calculated using Equation 5.4.4.

$$\overline{S}_L = \frac{LG}{2D_H} \quad (5.4.4)$$

where G is the tangent grade and D_H is the distance from high point to tangent point.

If Equation 5.4.4 is substituted into Equation 5.2.4 (to calculate Q_G) and the result is substituted into Equation 5.4.3 (with $Q_C = 0$), then Equation 5.4.5 gives the spacing for the first inlet on a crest vertical curve.

$$L = \left[\frac{24,394}{C i w n} \frac{d^{8/3}}{S_T} \right]^2 \frac{G}{2D_H} \quad (L < D_H) \quad (5.4.5)$$

After the initial spacing is calculated, the mean slope from Equation 5.3.9 is used with the depth of water, transverse slope and inlet geometry to determine the grate efficiency, carryover and bypass.

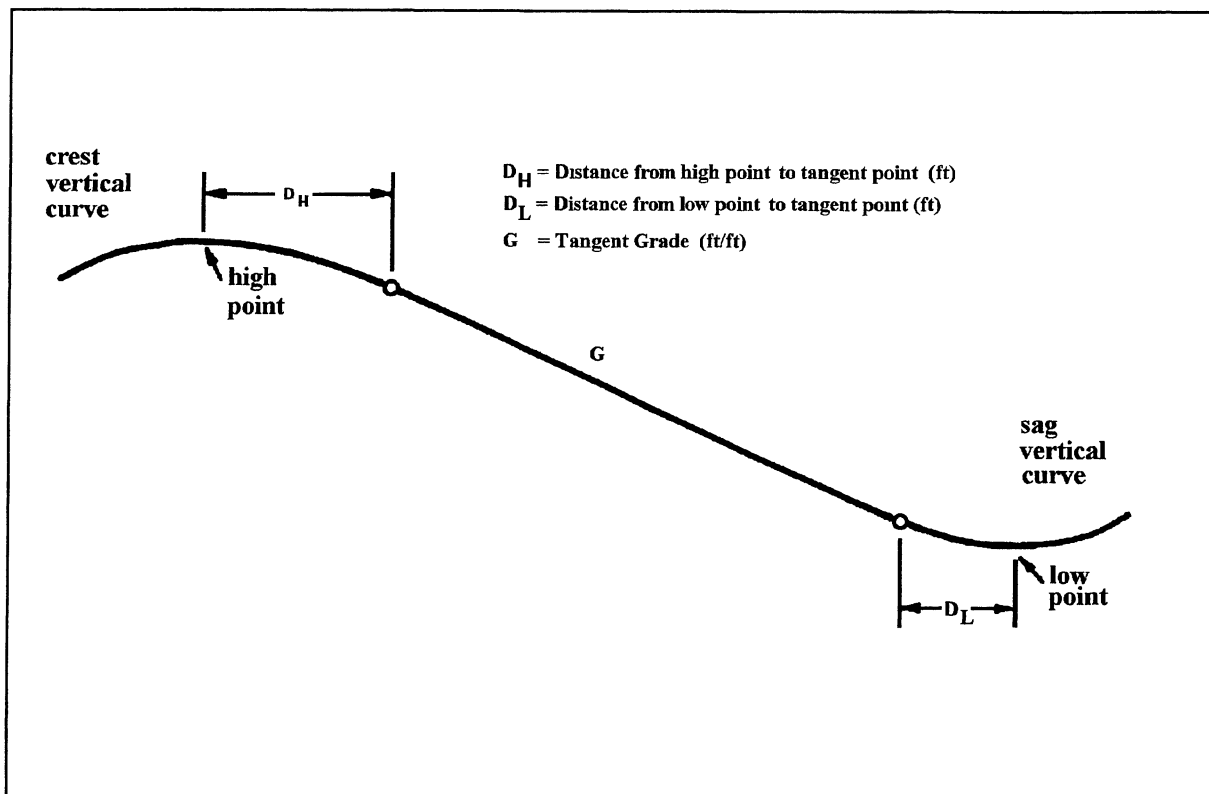


Figure 5.4.2
Grade Profile for a Hypothetical Roadway

The procedure for locating an inlet on a sag vertical curve is analogous to the crest vertical curve, except a safety factor of two is used to compensate for the high flooding potential of "valleys" and the likelihood of clogging. The mean slope is calculated by using Equation 5.4.6 and the spacing is found by using Equation 5.4.7.

$$\overline{S}_L = \frac{LG}{4D_L} \quad (5.4.6)$$

$$L = \left[\frac{24,394}{wCin} \frac{d^{8/3}}{S_T} \right]^2 \frac{G}{4D_L} \quad (L < D_L) \quad (5.4.7)$$

It is assumed that L falls within the distance D_H or D_L in Equations 5.4.5 and 5.4.7. If it does not, the mean slope of the street is simply calculated by weighting individual slopes between the break points over a given length of gutter, as shown in Figure 5.4.2. This mean slope is used to compute spacing as though the inlet is located on a constant grade.

Example 5.4.1 (Gutter Inlet Spacing - Rational Method)

Given: A 24' wide road without parking, shown on Figure 5.4.3, is located in Indianapolis. It has a longitudinal slope of 1.0 percent and a transverse slope of 3.0 percent. Assume that the street has an asphalt pavement with a smooth finish and that a 15 foot strip of the adjacent parkway contributes to the inlet.

Calculate: the spacing and location of the first two (2) gutter inlets, using the grate shown in Figure 5.3.2, design data presented in Section 5.3.4, and design calculation sheet in Figure 5.4.1.

Step 1: Determine the maximum flow in the street

Since the maximum top width can only be 4 feet, the depth of flow can be calculated as:

$$d = S_T T = (0.03)(4) = 0.12 \text{ feet}$$

From Table 4.2, $n = 0.013$

The maximum gutter flowrate is determined using Equation 5.2.4.

$$Q_G = \frac{0.56 (0.01)^{1/2}}{(0.013)(0.03)} (0.12)^{8/3} = 0.50 \text{ cfs}$$

Step 2: Using the depth and street slopes, calculate the grate capacity.

From Figure 5.3.2, $K = 16$, and from Equation 5.3.1:

$$Q_1 = K(d)^{5/3} = 16(0.12)^{5/3} = 0.47 \text{ cfs}$$

The bypass Q_b is 0.5 cfs - 0.47 cfs or 0.03 cfs. This will be passed to the second inlet.

Step 3: Compute a composite runoff coefficient

Assume a value C of 0.25 for the parkway. The composite coefficient for a 1 foot length of pavement is:

$$C = \frac{12(0.95) + 15(0.25)}{27} = 0.56$$

Step 4: Calculate the rainfall intensity

For the recommended 10-year return period and a five-minute time of concentration, the rainfall intensity for Indianapolis is found using the equation on page 2-22 from Example 2.2.3.

$$i = \frac{CT_r^\alpha}{(t + d)^\beta} \quad \cdot \quad i_5^{10} = \frac{2.1048 (10)^{0.1733}}{\left(\frac{5}{60} + 0.47\right)^{1.1289}} = 6.12 \text{ inches/hr}$$

Step 5: Calculate the spacing to the first inlet by using Equation 5.4.3

This is the required distance from the start of a subbasin to the first inlet.

$$L = \frac{43,560 (Q_i + Q_b - Q_c)}{Ci w} = \frac{(43,560) (0.50)}{(0.56) (6.12) (12+15)} = 235 \text{ feet}$$

This is the spacing for one side of the street only. If the same width of parkway contributed to both sides, then there would be an inlet at this location on the opposite side.

Step 6: Calculate the spacing to the next inlet

The total area contributing to the next inlet is reduced since this inlet must catch the carry-over of 0.03 cfs.

$$L = \frac{(43,560) (0.47 + 0.03 - 0.03)}{(0.56) (6.12) (12+15)} = 221 \text{ feet}$$

NOTE: This assumes that there will be a carry-over of 0.03 to the next inlet on this grade.

These calculations are also shown in Figure 5.4.4.

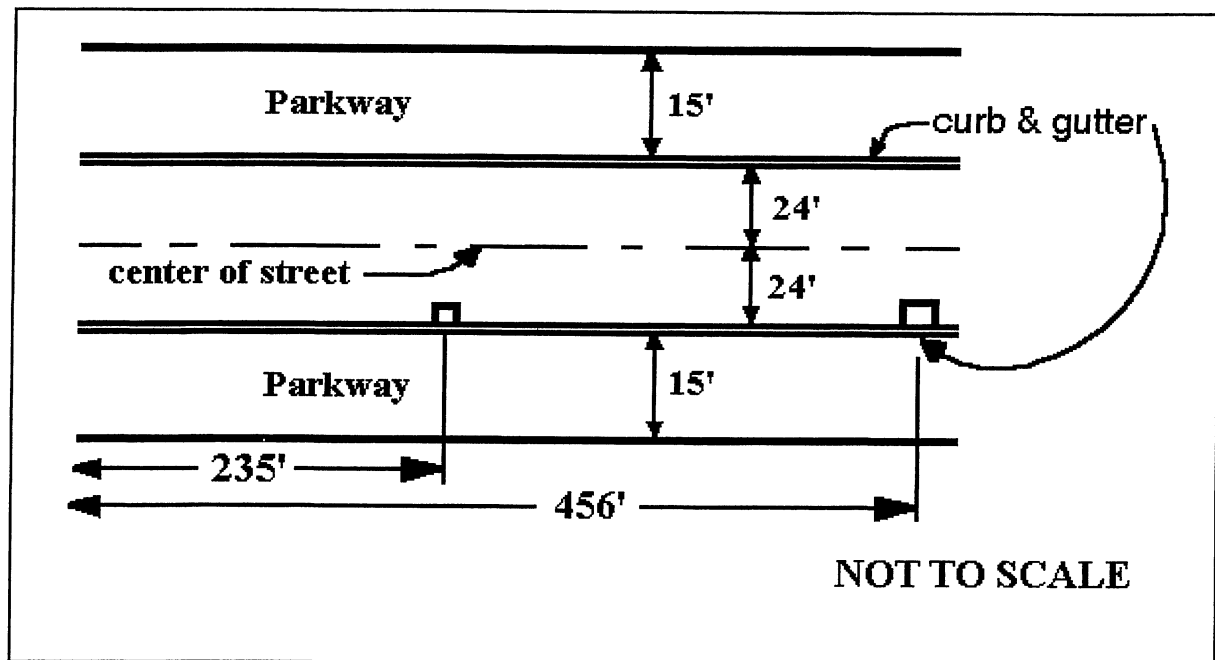


Figure 5.4.3
Depiction of Street Described in Example 5.4.1

[illegible]

Figure 5.4.4

5.5 - COMPUTER PROGRAMS

It should be obvious that the rather tedious calculations required for the sizing and location of gutters and inlets are well suited to the computer. A couple of computer programs which have been developed for solving problems similar to the examples in this chapter are listed below.

5.5.1 - HEC-12

The HEC-12 Drop Inlet Design and Analysis Program automates calculations for finding the interception capacity of roadway and median inlets. It will calculate the pavement spread and required length of the drop inlet to intercept all storm flow. HEC-12 uses the procedure outlined by the Federal Highway Administration's Hydraulic Engineering Circular No. 12, "Drainage of Highway Pavements." The peak discharge can be directly entered or the program can calculate it using the Rational Method.

The program handles the following inlet types: curb, grate, combination grate and curb, 4-inch bridge scupper, slotted drain, and grate in trapezoidal ditch. It can handle inlets on continuous grades or in sumps, analyze existing inlets, and add the discharge from one inlet to another. (U.S. DOT, 1984)

5.5.2 - Neenah Foundry Company's Inlet Grate Capacities

The Neenah Foundry Company has a computer program available for Inlet Grate Capacities. The program allows the user to evaluate grate efficiencies under differing flow conditions. Two separate sub-programs are present. One evaluates inlet capacities, where the parameters of flow, longitudinal and transverse slopes, plugging factor and grate selection are input. The program then displays the depth of flow onto the pavement, and the amount of flow captured by the individual grate being evaluated. The program has the capability of evaluating a compound gutter section where the transverse slope of the gutter is greater than the transverse slope of the pavement.

The second sub-program evaluates an inlet at low points, i.e. sump conditions. The free open area of the grate and the wetted perimeter are input and a graph of head vs. flow is generated for both weir and orifice flow (Neenah, 1990). The program and supporting documentation can be obtained from the Neenah Foundry Company, Product Engineering Department, P.O. Box 729, Neenah, Wisconsin, 54957; (414) 725-7000.

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Chapter 6

STORMWATER STORAGE

Chapter 6 - STORMWATER STORAGE

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Chapter 6 - STORMWATER STORAGE

LIST OF PARAMETERS

A	Area (acres or hectares)
A'	Cross-sectional area of soil (ft ² or m ²)
A _D	Contributing developed area
A _o	Area of the orifice (ft ² or m ²)
A _T	Total contributing area (acres or hectares)
A _U	Contributing undeveloped area
C _d	Coefficient of discharge
C _D	Developed runoff coefficient
C _U	Undeveloped runoff coefficient
d	Depth (ft or m)
D	Pipe diameter (ft or m)
ΔD	Rainfall duration
g	Acceleration due to gravity (32.2 ft/sec ² or 9.81 m/sec ²)
h _L	Head loss or the gradient over a flow distance L (ft or m)
h _o	Height of water above the orifice (ft or m)
h _p	Height of water above the pipe opening (ft or m)
h _w	Height of water above the weir opening (ft or m)
i _d	Rainfall intensity at time t _d (in/hr or cm/hr)
i _U	Undeveloped rainfall intensity (in/hr or cm/hr)
I(t)	Inflow at time t (cfs or m ³ /sec)
K	Coefficient of permeability
K _e	Entrance loss coefficient
K _o	Outlet loss coefficient
L	Length (ft or m)
n	Manning's roughness coefficient
O(t)	Outflow or runoff (cfs or m ³ /sec) or (cfs/acre or m ³ /sec / hectare)
P(t)	Accumulated rainfall (inches)
Q	Infiltration flows (cfs or m ³ /sec)
Q _U	Peak undeveloped flow (cfs or m ³ /sec)
R(t)	Direct surface runoff (inches or cm)
q _p	Peak flow (cfs or m ³ /sec)
Q _I (t)	Magnitude of the inflow hydrograph at time t
Q _O (t)	Magnitude of the outflow hydrograph at time t
S	Ultimate abstraction
S(t _d)	Required storage (acre-ft)
ΔS	Change in storage (ft ³ or m ³)
S _{max}	Required detention storage
t'	Time at which inflow is equal to outflow
t _b	Time of base
t _c	Time of concentration

LIST OF PARAMETERS (cont'd)

t_d	Storm duration
t_p	Time to peak
t_t	Time of travel (hrs)
Δt	Time interval (sec)
v	Velocity (ft/sec or m/sec)
V	Volume (ft ³ or m ³)
$V_i(t_d)$	Inflow volume
$V_o(t_d)$	Outflow volume

Chapter 6 - STORMWATER STORAGE

6 1 - INTRODUCTION

In Chapter 3, the response of an "urbanized" watershed to a storm was shown to be considerably different from that of rural, undeveloped watershed. Land which once allowed infiltration of rainwater is covered with impervious parking lots, streets, and buildings or homes, thereby causing a higher percentage of rainfall to become surface runoff. Improvements such as streets, curbs, and storm sewers collect and convey this runoff to its point of disposal more rapidly than in an undeveloped watershed, resulting in a higher runoff volume in a shorter period of time.

These increased runoff rates produced by the urbanization of a watershed may overtax succeeding portions of the larger drainage network causing flooding problems downstream. To circumvent this difficulty, storage facilities are provided to receive the runoff from the developed watershed while releasing water to the larger drainage network at a reduced rate. This reduced rate is determined by using parameters fixed by local ordinance or by calculating the available capacity of the downstream conveyance system.

Various types of storage facilities used and the methods employed in computing required storage volumes are discussed in this chapter. Design considerations are outlined and example problems are included to illustrate applications.

6 1 1 - Types of Storage Facilities

Storage facilities can be separated into two general categories detention and retention

Detention storage involves detaining or slowing runoff and then releasing it. A *detention basin* has a positive outlet that completely empties all runoff between storms. In some situations, the excavation of a detention facility may extend below the water table or outlet level where the bottom is sealed by sedimentation. This case is referred to as a *detention pond* or *wet bottom detention basin*. The detention pond also has a positive outlet and releases all temporary storage.

Retention facilities retain runoff for an indefinite amount of time and have no positive outlet. Runoff is removed only by infiltration through a porous bottom or by evaporation. Retention ponds and lakes are examples of retention facilities that may be built in a development and in many cases, enhance the overall project.

Both types of storage are very common, although the designed retention becomes less practical as the size of the drainage area increases. Retention ponds may need 2 to 7 times

more permanent pool volume than the temporary storage volume needed for a detention basin (Maidment, 1993). The use and design of these facilities are outlined in Section 6.3.

6 2 - COMPUTATION OF STORAGE VOLUME

As previously stated, storage facilities are designed to receive the stormwater collected by a drainage system for a development and release it at a specified rate which may be zero for retention facilities. The primary goal is to provide the storage volume necessary to offset the impacts of urbanization. If infiltration and evaporation are neglected during the period of runoff, the continuity equation for a detention pond is written as:

$$I(t) - O(t) = \frac{\Delta S}{\Delta t} \quad (6.2.1)$$

where $I(t)$ is the inflow to the pond from the sewer network at time (t) (cfs), $O(t)$ is the outflow from the pond into the larger drainage network at time (t) (cfs), and ΔS is the change in storage (ft^3) in time interval $(\Delta t)(\text{sec})$

Equation 6 2 1 may also be written as:

$$(I_1 + I_2) \frac{\Delta t}{2} - (O_1 + O_2) \frac{\Delta t}{2} = S_2 - S_1 \quad (6.2.2)$$

where subscripts 1 and 2 denote the flows, volumes, and storage at times t_1 and t_2 and $\Delta t = t_1 - t_2$

When the inflow and outflow hydrographs are known, the largest value of $S_2 - S_1$ found in Equation 6 2 2 is the required storage. Normally the hydrographs are not known in the design phase and are usually estimated. The following is a discussion of some of the methods used to estimate the storage in the design of detention facilities. For retention facilities the value of $O(t)$ is equal to the sum of the evaporation and infiltration rates. During a storm these may be negligible and the required storage volume is therefore equal to the runoff volume

6 2 1 - Rational Method

The rational method discussed in Section 3 2, computes a peak runoff rate only; however, it may be extended to compute volumes by multiplying the peak flow rate by the storm duration

The peak flow rate permitted to leave the detention pond, $O(t)$, is based on the contributing

undeveloped area, A_U , undeveloped runoff coefficient, C_U , and rainfall intensity, i_U , associated with the time of concentration of the undeveloped basin. The return period for the rainfall intensity is fixed by local ordinance or by the design parameters of the larger downstream drainage network. In some cases, the allowable outflow may be even more limited due to the capacity of the succeeding network. Often this release rate is expressed in terms of cfs/acre and the rates range from 0.10 - 0.25. In either case, the outflow is assumed to remain constant for all storm durations, t_d . Therefore the volume at t_d is simply the product of $O(t)$ and t_d . This is illustrated in Figure 6.2.1 where the line $V_O(t_d)$ is the outflow volume vs. time.

The inflow rate, $I(t)$, is calculated by using the contributing developed area, A_D , the developed runoff coefficient, C_D , and a rainfall intensity, i_d , corresponding to storm duration, t_d , and the return period for the detention pond design. Thus, for various durations, the peak flow and therefore the volume of runoff $V_I(t_d)$ may be computed. Figure 6.2.1 shows the curve of $V_I(t_d)$ vs t_d .

The maximum difference between the inflow and outflow volumes is the required detention pond storage. This is shown in Figure 6.2.1 as S_{\max} .

The method may also be expressed as.

$$S(t_d) = (C_D i_d A_D - C_U i_U A_U) \frac{t_d}{12} \quad (6.2.3)$$

where $S(t_d)$ is the required storage (acre-ft), C_D is the developed runoff coefficient, C_U is the undeveloped runoff coefficient, A_D is the area of developed watershed (acres), A_U is the area of undeveloped watershed (acres), i_d is the rainfall intensity (inches/hr) for the return period of the detention pond and duration t_d , i_U is the rainfall intensity corresponding to the time of concentration of the undeveloped watershed and a return period based upon either local ordinance or the capacity of the downstream system (inches/hr), and t_d is the storm duration which is varied to find the maximum peak storage (hrs).

The factor 1/12 in Equation 6.2.3 is used to convert cfs-hours to acre-feet.

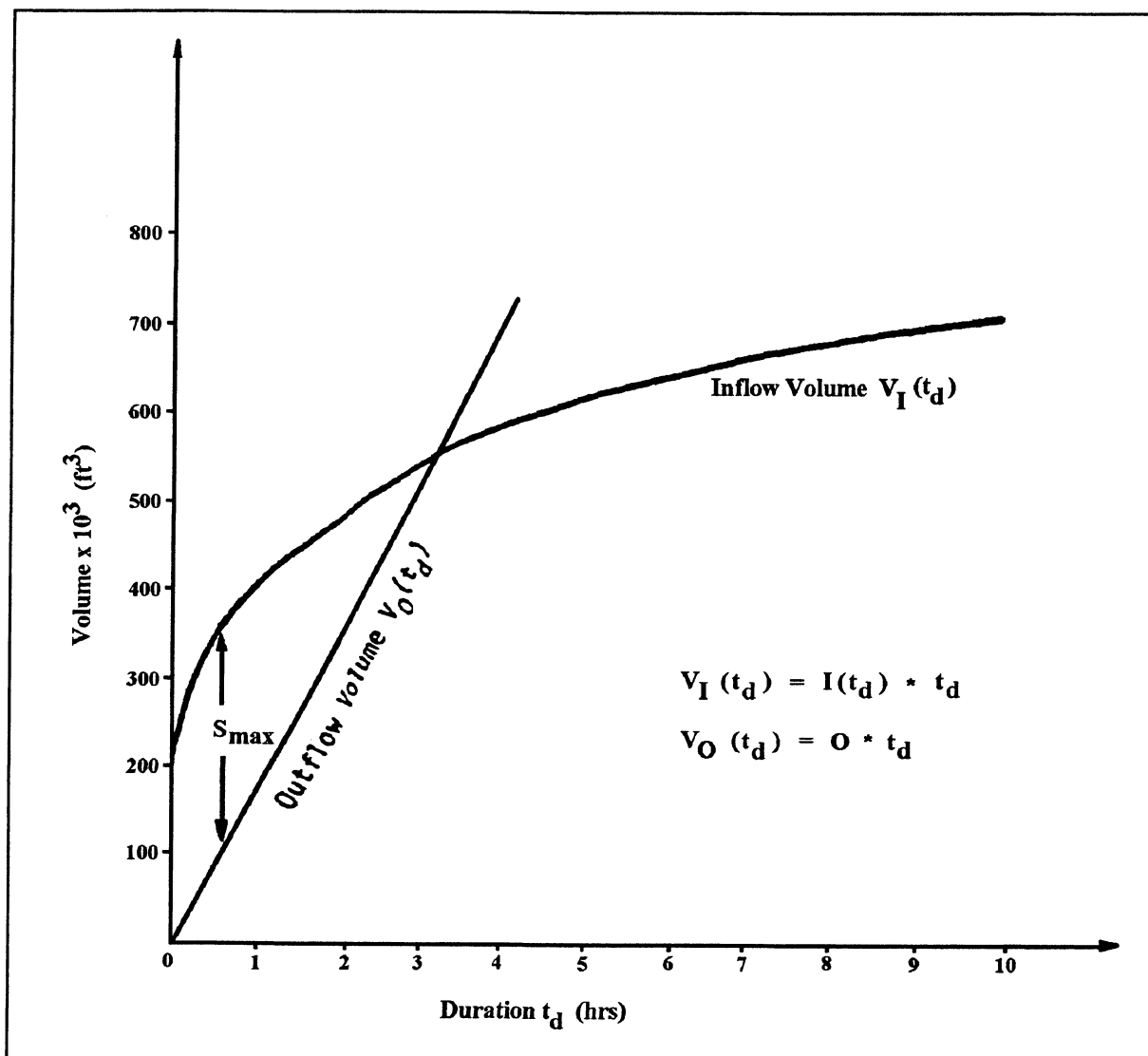


Figure 6.2.1
Graphical Representation of Storage Volumes as Determined by the Rational Method

As in the graphical approach, various storm durations, t_d , are selected and the largest value of $S(t_d)$ obtained is used to design the detention pond. Figure 6.2.2 is a worksheet which may be used for computations based on Equation 6.2.3.

The step by step procedure to compute the storage volume by using the rational method is as follows.

- Step 1: Determine the area, A_U , runoff coefficient, C_U , and time of concentration for the undeveloped site. By using the appropriate intensity-duration-frequency curve determine the intensity, i_U , corresponding to the return period for the allowable outflow rate
- Step 2. Calculate the runoff ($O(t)$) from the undeveloped site. $O(t) = C_U i_U A_U$; or use the allowable release rate determined by other methods.
- Step 3 Determine the developed runoff coefficient, C_D
- Step 4: Determine the rainfall intensities (i_d) for various durations (t_d), for the specified return period. Recommended durations are 10, 20, 30, 40, 50 min and 1, 1.5, 2, 3, 4, 5, 6, 7, 8, 9 and 10 hours
- Step 5 Determine the inflow rate to the detention pond $I(t_d) = C_D i_d A_D$
- Step 6 Compute the required storage for each duration,

$$S(t_d) = \frac{t_d}{12} [I(t_d) - O(t)] \quad (\text{acre-feet})$$

- Step 7 Select the largest volume

Various agencies have set guidelines for the selection of i_U , i_D , C_U and C_d . One agency, the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) (Metropolitan Sanitary District of Greater Chicago prior to 1989), uses the following criteria.

- i_U based on a 3-year return period
- i_D based on 100-year return period
- $C_U \leq 0.15$
- $C_D \geq 0.35$

In Example 6.2.1 these criteria are applied and the use of this procedure is demonstrated. A computer program which can be used to perform these computations is included at the end of the chapter. The calculations may also be performed conveniently using a spreadsheet.

Project _____ Detention Facility Design Return Period _____ yrs.

Designer _____ Release Rate Return Period _____ yrs.

Watershed Area _____ acres

Time of Concentration (undeveloped watershed) _____ minutes

Rainfall Intensity (i_U) _____ inches/hr

Undeveloped Runoff Coefficient (C_U) _____

Undeveloped Runoff Rate ($O = C_U i_U A_U$) _____ cfs

Developed Runoff Coefficient (C_D) _____

Storm Duration t_d (hrs)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $I(t_d)$ $(C_D i_d A_D)$ (cfs)	Outflow Rate O $(C_U i_U A_U)$ (cfs)	Storage Rate $I(t_d) - O$ (cfs)	Required Storage $\left[I(t_d) - O \right] \frac{t_d}{12}$ (acre-ft)
0.17					
0.33					
0.50					
0.67					
0.83					
1.0					
1.5					
2.0					
3.0					
4.0					
5.0					
6.0					
7.0					
8.0					
9.0					
10.0					

Figure 6.2.2

Computation Sheet for Detention Storage Calculations Using the Rational Method

Example 6.2.1

A 10-acre parcel of land located in Lafayette, Indiana which is currently a flat pasture with a clay and silt loam soil is to be developed to an area with the following characteristics: 20% parks, 50% single-family homes and 30% business. The overland flow path has a length of 150 feet and slope of 0.01 ft/ft. Determine the size of the detention pond required so that the 100-year runoff from the developed land will not exceed the 3-year runoff of the undeveloped land (MWRDGC guidelines). Use the rational method procedure outlined in Section 6.2.1.

Step 1 · Undeveloped runoff A = 10 acres

From Table 3.2.1, the runoff coefficient is 0.30. However MWRDGC requires that the undeveloped runoff coefficient be less than or equal to 0.15. The overland flow time of concentration is computed by Kerby's Equation in Table 3.2.4.

$$t_c = 0.83 \left[\frac{(150)(0.40)}{\sqrt{0.01}} \right]^{0.467}; \quad t_c = 16.46 \text{ minutes}$$

Step 2 · The peak undeveloped flow is computed by the rational method. Following the procedure outlined in Section 2.2.5, the intensity for a three-year return period and 16.46 minute duration for Lafayette, Indiana is found as follows.

$$p_1^{10} = 1.95" \quad p_{24}^{10} = 4.18" \quad p_1^{100} = 2.73"$$

$$p_1^{10} / p_{24}^{10} = 0.466 \quad X = p_1^{100} / p_1^{10} = 1.40$$

$$\text{Using Figure 2.2.3} \quad a_1 = 28.6 \quad d' = 9.2 \quad n' = 0.78$$

$$a' = a_1 i_1^{10} \log(10^{(2-X)} T_r^{(X-1)}) = (28.6)(1.95) \log(10^{(0.60)} 3^{(0.40)}) = 44.11$$

$$i_t^{T_r} = \frac{a'}{(60t + d')^{n'}} \quad \therefore i_{16.46/60}^3 = \frac{44.11}{(16.46 + 9.2)^{0.78}} = 3.51 \text{ inches/hr}$$

$$Q_U = (0.15)(3.51)(10) = 5.27 \text{ cfs}$$

Step 3 · Compute the developed runoff coefficient. A composite runoff coefficient is determined by using the information in Table 3.2.2.

	<i>Acres</i>	<i>Type of Cover</i>	<i>Runoff Coefficient</i>	$C_n A_n$
	2	Parks	0.17	0.34
	5	Single Family Homes	0.40	2.00
	3	Business	0.60	1.80
TOTAL	10			4.14

$$C_D = \frac{\Sigma C_n A_n}{A_T} = \frac{4.14}{10} = 0.41 > 0.35$$

Step 4 Determine rainfall intensities for a 100-year return period for duration of 10, 20, 30, 40 and 50 minutes and 1, 1.5, 2, 3, 4, 5, 6, 7, 8, 9 and 10 hours for Lafayette, Indiana using the following equation (where t is in hours):

$$i_t^{100} = \frac{78.08}{(60t + 9.2)^{0.78}}$$

<i>t</i>	<i>i (inches/hr)</i>	<i>t</i>	<i>i (inches/hr)</i>
10 minutes	7.79	3 hours	1.31
20 minutes	5.62	4 hours	1.05
30 minutes	4.46	5 hours	0.89
40 minutes	3.74	6 hours	0.78
50 minutes	3.24	7 hours	0.69
1 hour	2.87	8 hours	0.62
1.5 hours	2.16	9 hours	0.57
2 hours	1.76	10 hours	0.53

Step 5 Determine the inflow rate, $I(t_d)$, and required storage at each of the above durations (See Figure 6.2.3)

The peak required storage occurs at a 40 minute duration and is found to be 0.56 acre-feet.

Project Example 6.2.1 Detention Facility Design Return Period 100 yrs.

Designer TTB Release Rate Return Period 3 yrs.

Watershed Area 10 acres

Time of Concentration (undeveloped watershed) 16.46 minutes

Rainfall Intensity (i_U) 3.51 inches/hr

Undeveloped Runoff Coefficient (C_U) 0.15

Undeveloped Runoff Rate ($O = C_U i_U A_U$) 5.27 cfs

Developed Runoff Coefficient (C_D) 0.41

Storm Duration t_d (hrs)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $I(t_d)$ $(C_D i_d A_D)$ (cfs)	Outflow Rate O $(C_U i_U A_U)$	Storage Rate $I(t_d) - O$ (cfs)	Required Storage $\left[I(t_d) - O \right] \frac{t_d}{12}$ (acre-ft)
0.17	7.79	31.94	5.27	26.67	0.38
0.33	5.62	23.04	5.27	17.77	0.49
0.50	4.46	18.29	5.27	13.02	0.54
0.67	3.74	15.33	5.27	10.06	0.56
0.83	3.24	13.28	5.27	8.01	0.55
1.0	2.87	11.77	5.27	6.50	0.54
1.5	2.16	8.86	5.27	3.59	0.49
2.0	1.76	7.22	5.27	1.95	0.33
3.0	1.31	5.37	5.27	0.10	0.03
4.0	1.05	4.31	5.27	*	*
5.0	0.89	3.65	5.27	*	*
6.0	0.78	3.20	5.27	*	*
7.0	0.69	2.83	5.27	*	*
8.0	0.62	2.54	5.27	*	*
9.0	0.57	2.34	5.27	*	*
10.0	0.53	2.17	5.27	*	*

Peak
Storage

* Since $I(t_d) - O < \text{zero}$, there is no storage needed.

Figure 6.2.3
Detention Storage Calculations for Example 6.2.1 Using the Rational Method

Under certain conditions it may be impossible to convey and collect all the runoff from a given watershed. The result is that some runoff is discharged directly into the larger drainage network without being detained. To compensate for this unrestricted release, the allowable release rate, $O(t)$, is lessened by that amount.

A shortcoming of this procedure is that the release rate is a constant and does not account for changing water surface elevations within the facility which in turn increases the head above and therefore the discharge through a restrictor. The storage volume determined will be smaller than that actually needed. A more accurate procedure is to employ a storage routing program which requires a stage-storage-discharge curve. Chapter 8 presents two computer programs that perform an iterative routing procedure that is more precise for simulating the inflows and outflow and the corresponding storage volume.

6 2 2 - Soil Conservation Service Hydrograph Method

In Section 3.3, methods were presented by which the time distribution of stormwater runoff could be found by using rainfall data and certain basin parameters. These hydrographs, developed by the Soil Conservation Service, are easily used to calculate storage requirements for stormwater storage facilities.

As previously described, the difference between the inflow from the developed watershed and the allowable outflow from a detention pond, is the required storage volume. The outflow is determined by using characteristics of the undeveloped watershed and a rainfall frequency equal to or less than the receiving system can handle, or as prescribed by local ordinance.

Inflow and outflow hydrographs are shown in Figure 6.2.4, in which the difference between the hydrographs, which is the required storage, is shown shaded. At point C, the inflow rate to the detention pond is equal to the outflow rate (e.g., the pond will start to empty). The outflow rate is assumed to remain constant in this figure but actually this rate will depend upon the depth of water in the pond and the type of outlet structure (i.e. weir, orifice or pipe-flow). It should also be noted that for a detention pond the total inflow volume is equal to the total outflow volume.

The inflow hydrograph can be determined using the procedure outlined in Chapter 3. The outflow hydrograph is found by using the depth-storage relationship for the outflow structure. Using this information, the required storage volume is found by calculating the area between the hydrographs. This is found by using a planimeter, digitizer, or by using Equation 6 2 5,

$$S(t_d) = \frac{\Delta t}{12} \sum_{t=\Delta t}^{t'} [Q_I(t) - Q_O(t)] \quad (6.2.5)$$

where $Q_I(t)$ is the magnitude of the inflow hydrograph at time t , $Q_O(t)$ is the magnitude of the

outflow hydrograph at time t , Δt is the time interval used in calculations, t' is the time at which inflow = outflow (point C in Figure 6.2.4), and $S(t_d)$ is the total required storage (acre-feet)

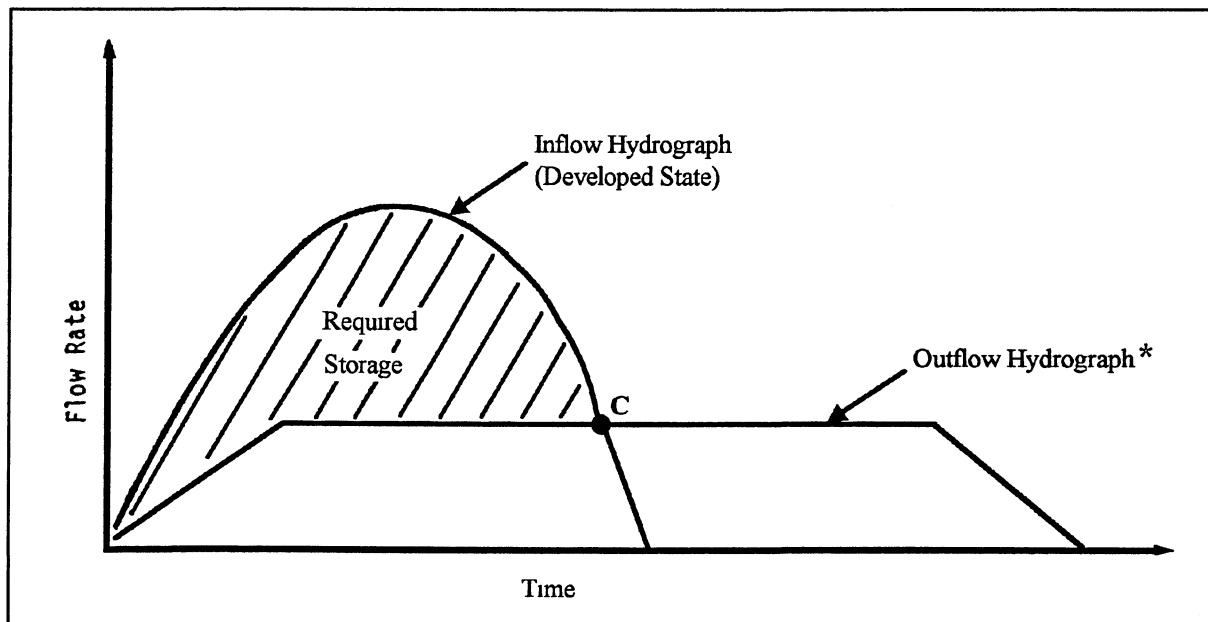


Figure 6.2.4
Inflow and Outflow Hydrographs for a Hypothetical Detention Pond

* Assumes a constant release rate. See Chapter 8 for computer applications with outflows dependent on pond stage.

An outline of the procedure used in determining the required storage with the hydrograph method is as follows

- Step 1 Calculate the curve numbers for the developed and undeveloped basins.
- Step 2 Find the times of concentration, t_c , for the basin as outlined in Section 3.3.
- Step 3: From t_c , calculate ΔD , q_p and t_p using Equations 3.4.2 and 3.4.3, respectively, for the developed and undeveloped site.
- Step 4 Determine the coordinates of the inflow and outflow unit hydrographs by using Figure 3 4 3 for a dimensionless unit hydrograph or by plotting a triangular unit hydrograph

- Step 5: From the design storm duration, depth, time distribution and frequency, calculate the cumulative rainfall at time interval ΔD for both the undeveloped and developed states.
- Step 6: Using the basin curve number and ultimate abstraction, S , calculate the cumulative runoff, $R(t)$ at each time interval ΔD by using the rainfall data from Step 5.
- Step 7: Using Equation 3.4.8 or by summing up the coordinates for a triangular hydrograph, calculate the storm hydrographs.
- Step 8: Using the peak flow from the undeveloped site as the peak outflow, calculate the outflow hydrograph as determined by the type of outflow structure.
- Step 9: Calculate the required storage using Equation 6.2.5, using a planimeter, or digitizer.

The application of this method is illustrated in Example 6.2.2.

Example 6.2.2

For the 150 acre watershed in Indianapolis, Indiana described below, determine the required storage volume by using the hydrograph method described in Section 6.2.2. The inflow is to be based on a 100-year, 1-hour storm and the outflow is to be restricted to a 10-year, 1-hour storm. The soil type is C. Use a triangular unit hydrograph and AMC II.

Undeveloped

75 acres Meadow (good condition)
45 acres Woods (good cover)
30 acres Pasture (good condition)

Travel Path of 5625 ft. over
unpaved land with a 0.6% slope

Developed

50 acres Residential (1/2 acre lots)
10 acres Gravel road
40 acres Parking lots
50 acres Lawns (fair condition)
Travel Path of 3060 ft. over a paved area
with a slope of 1% and 9000 ft. of 24 in.
storm sewer with a slope of 3% (assume pipe
is flowing full and $n = 0.013$).

Step 1: Determine a composite curve number for both states.

<i>UNDEVELOPED</i>			
	CN	AREA	$CN_n \times A_n$
Meadow (good condition)	71	75	5325
Woods (good cover)	70	45	3150
Pasture (good condition)	74	30	2220
Total		150	10,695

<i>DEVELOPED</i>			
	CN	AREA	$CN_n \times A_n$
Residential (1/2 acre)	80	50	4000
Gravel roads	89	10	890
Parking Lots	98	40	3920
Lawns (fair condition)	79	50	3950
Total		150	12,760

$$CN_U = \frac{\sum [CN_n \times A_n]}{A_T} = \frac{10,695}{150} = 71.3$$

$$CN_D = \frac{\sum [CN_n \times A_n]}{A_T} = \frac{12,760}{150} = 85.1$$

Step 2 Determine the time of concentration for both states.

Undeveloped: From Figure 3.4.5 with a slope of 0.006 and unpaved area as the cover the velocity is found to be 1.25 ft/sec. The time of concentration is computed by:

$$t_c = \frac{L}{3600v} = \frac{5625}{3600(1.25)} = 1.25 \text{ hrs.}$$

Developed The time of concentration is the sum of the overland travel time and the channel travel time.

The overland travel time is found using Figure 3.4.5 with a 1% slope and a paved area surface (sheet flow)
 $V = 2.0 \text{ ft/sec}$

$$t_t = \frac{L}{3600v} = \frac{3060}{3600(2.0)} = 0.43 \text{ hrs.}$$

The channel travel time is computed by calculating the velocity from Manning's formula (Equation 4.2.1) for full pipe flow.

$$v = \frac{Q}{A} = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.013} \left[\frac{2}{4} \right]^{2/3} [0.03]^{1/2} = 12.5 \text{ ft/sec.}$$

$$t_t = \frac{L}{3600v} = \frac{9000}{(3600)(12.5)} = 0.20 \text{ hrs}$$

$$t_c = 0.20 \text{ hrs} + 0.43 \text{ hrs} = 0.63 \text{ hrs.}$$

Step 3 Calculate ΔD , t_p and q_p ($A = 150$ acres $= 0.234$ mi²)

Undeveloped: $\Delta D = (0.133)t_c = (0.133)(1.25) = 0.17$ hrs (10 minutes)

$$t_p = \frac{\Delta D}{2} + 0.6 t_c = \frac{0.17}{2} + 0.6 (1.25) = 0.84 \text{ hrs}$$

$$q_p = \frac{484AQ}{T_p} = \frac{484 (0.234) (1)}{0.84} = 134.83 \text{ cfs}$$

Developed: $\Delta D = (0.133)(0.63) = 0.08$ hrs (5 minutes)

$$t_p = \frac{0.08}{2} + 0.6 (0.63) = 0.42 \text{ hrs}$$

$$q_p = \frac{(484) (0.234) (1)}{0.42} = 269.66 \text{ cfs}$$

Step 4 The coordinates of the triangular unit hydrographs are found by plotting t_p , q_p and t_b as shown in Figure 6.2.5.
The base time, $t_b = 2.67 t_p$

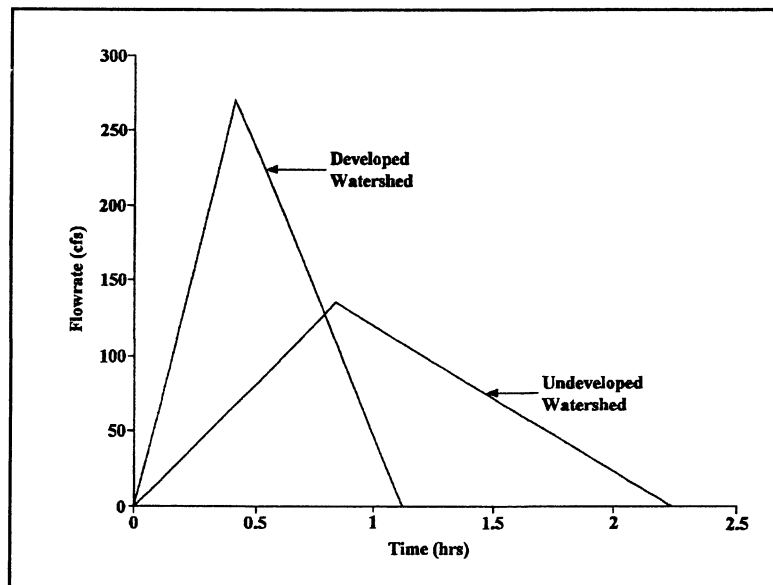


Figure 6.2.5
Triangular Unit Hydrographs for Example 6.2.2

Step 5: Use the Huff median (50% probability) first quartile distribution for Indianapolis (Table 2.1.5) and calculate the cumulative rainfall at 10-minute intervals for the undeveloped site and 5-minute intervals for the developed site.

The undeveloped 10-year, 1-hour depth and the developed 100-year, 1-hour depth for Indianapolis is determined from Equation 2.2.16

$$i_1^{10} = \frac{56.33}{60(1) + 9.2)^{0.78}} = 2.07 \text{ inches}$$

$$i_1^{100} = \frac{78.08}{60(1) + 9.2)^{0.78}} = 2.87 \text{ inches}$$

Cum. % of Storm Time	Time (min)	Percent of Total Storm Rainfall			Increment of Rainfall (in)	
		Cumulative	Incremental		Undeveloped.	Developed
			(10 min)	(5 min)	Total=2.07"	Total=2.87"
0	0	0	-	-	-	-
8.34	5	17		17		0.49
16.67	10	32	32	15	0.66	0.43
25.00	15	48		16		0.46
33.33	20	57	25	9	0.52	0.26
41.67	25	63		6		0.16
50.00	30	69	12	6	0.25	0.16
58.33	35	75		6		0.16
66.67	40	81	11	6	0.23	0.16
75.00	45	87		6		0.16
83.33	50	91	11	6	0.23	0.16
91.67	55	96		5		0.15
100.00	60	100	9	4	0.18	0.12

Step 6: Use the curve numbers for both the developed and undeveloped conditions and calculate the cumulative runoff at each interval. First find the ultimate abstraction for both conditions

$$S = \frac{1000}{CN} - 10$$

$$S = \frac{1000}{71.3} - 10 = 4.03 \text{ inches} \quad (\text{undeveloped})$$

$$S = \frac{1000}{85.1} - 10 = 1.75 \text{ inches} \quad (\text{developed})$$

Undeveloped

Time (min)	Incremental Rainfall (inches)	Cumulative Rainfall P(t) (inches)	Cumulative Runoff R(t) (inches)	Incremental Runoff R(t) (inches)
0		0	0	
10	0.66	0.66	0	0
20	0.52	1.18	0.03	0.03
30	0.25	1.43	0.08	0.05
40	0.23	1.66	0.15	0.07
50	0.23	1.89	0.23	0.08
60	0.18	2.07	0.30	0.07

$$R(t) = \frac{(P(t) - 0.2S)^2}{P(t) + 0.8S} = \frac{(P(t) - 0.81)^2}{P(t) + 3.22}$$

Developed

Time (min)	Incremental Rainfall (inches)	Cumulative Rainfall (inches)	Cumulative Runoff R(t) (inches)	Incremental Runoff R(t) (inches)
0		0	0	
5	0.49	0.49	0.01	0.01
10	0.43	0.92	0.14	0.13
15	0.46	1.38	0.38	0.24
20	0.26	1.64	0.55	0.17
25	0.16	1.80	0.66	0.11
30	0.16	1.96	0.77	0.11
35	0.16	2.12	0.89	0.12
40	0.16	2.28	1.01	0.12
45	0.16	2.44	1.14	0.13
50	0.16	2.60	1.27	0.13
55	0.15	2.75	1.39	0.12
60	0.12	2.87	1.49	0.10

$$R(t) = \frac{(P(t) - 0.35)^2}{P(t) + 1.40}$$

Step 7 Multiply the coordinates of the triangular unit hydrographs by each incremental runoff and compute the storm hydrograph by summing up the individual hydrographs. The storm hydrographs and construction are shown in Figures 6.2.6 and 6.2.7.

Step 8 The peak flow for the undeveloped site is found to be about 36 cfs. This is the maximum flowrate for the outflow hydrograph.

Step 9 For simplicity, assume that the outflow devices will regulate the flow such that a constant flow of 36 cfs will be released. Chapter 8 presents examples of the computer models available to simulate variable outflows. Calculate the

volume between the hydrographs as shown in Figure 6.2.8. The calculations are shown in Figure 6.2 8

Step 10. From Figure 6 2.8, the required storage is found to be 13.3 acre-feet.

Step 11 For a detention pond with a 400' x 400' bottom and 3:1 side slopes, the relationship between volume and pond depth can be determined as shown below

$$V = d \left[\frac{2(400 + 6d) + (400 \times 400)}{2} \right]$$

Solving by trial and error to find a depth which produces a volume of 579,780 ft³ gives d = 7.3 ft This gives the surface area of the pond as approximately 444 ft x 444 ft.

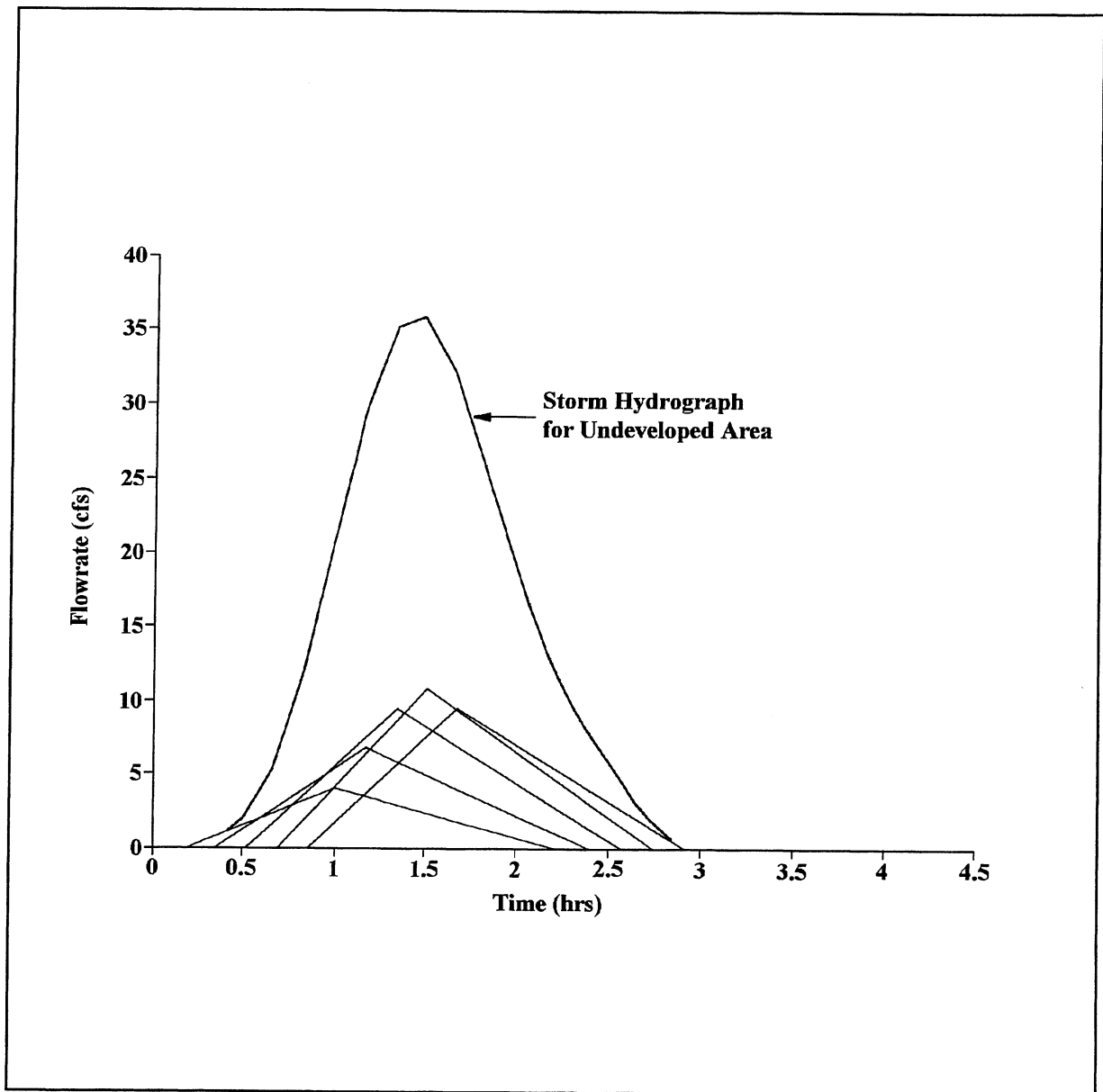


Figure 6.2.6
Storm Hydrograph for Undeveloped Site for Example 6.2.2

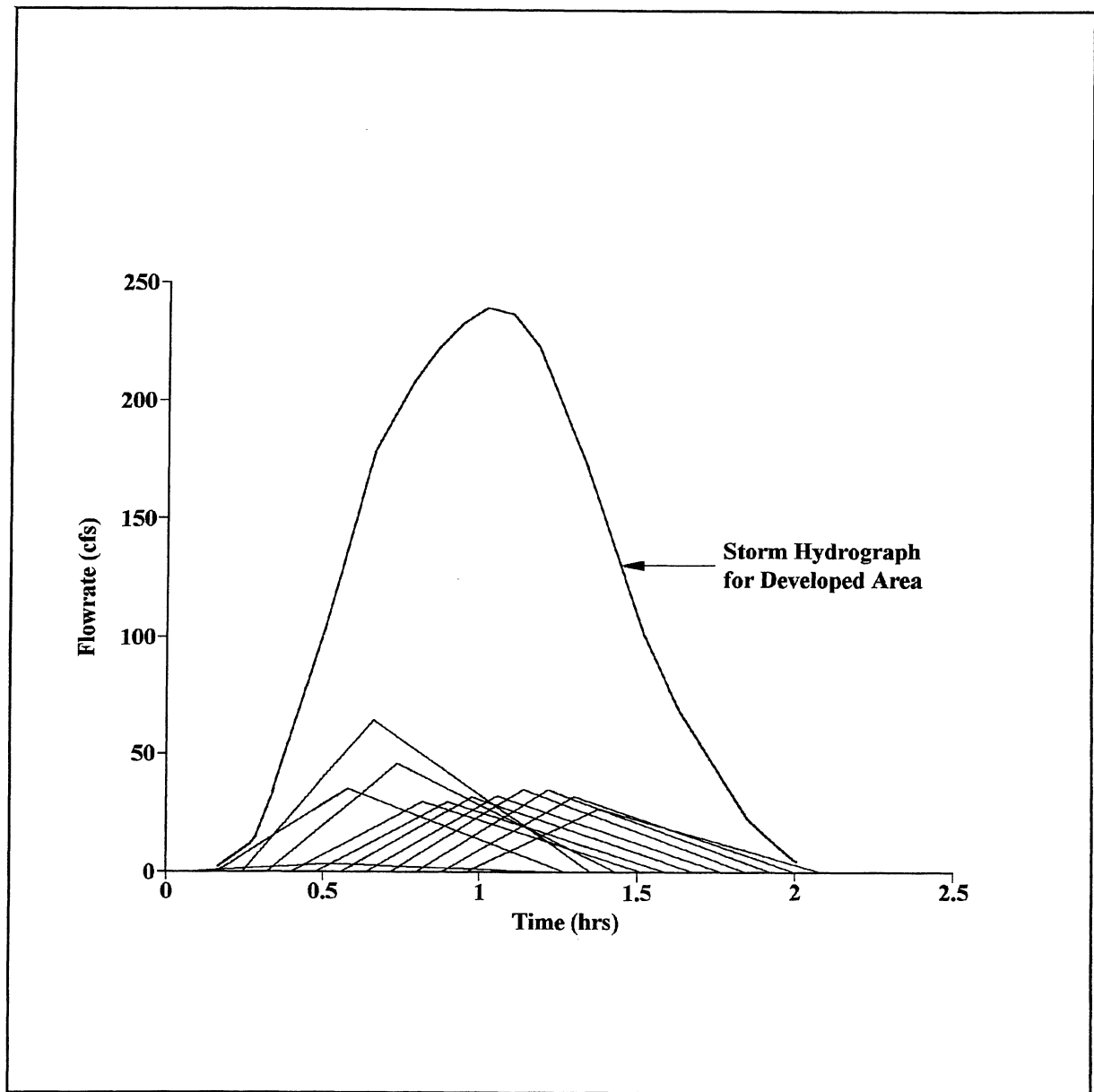


Figure 6.2.7
Storm Hydrograph for Developed Site for Example 6.2.2

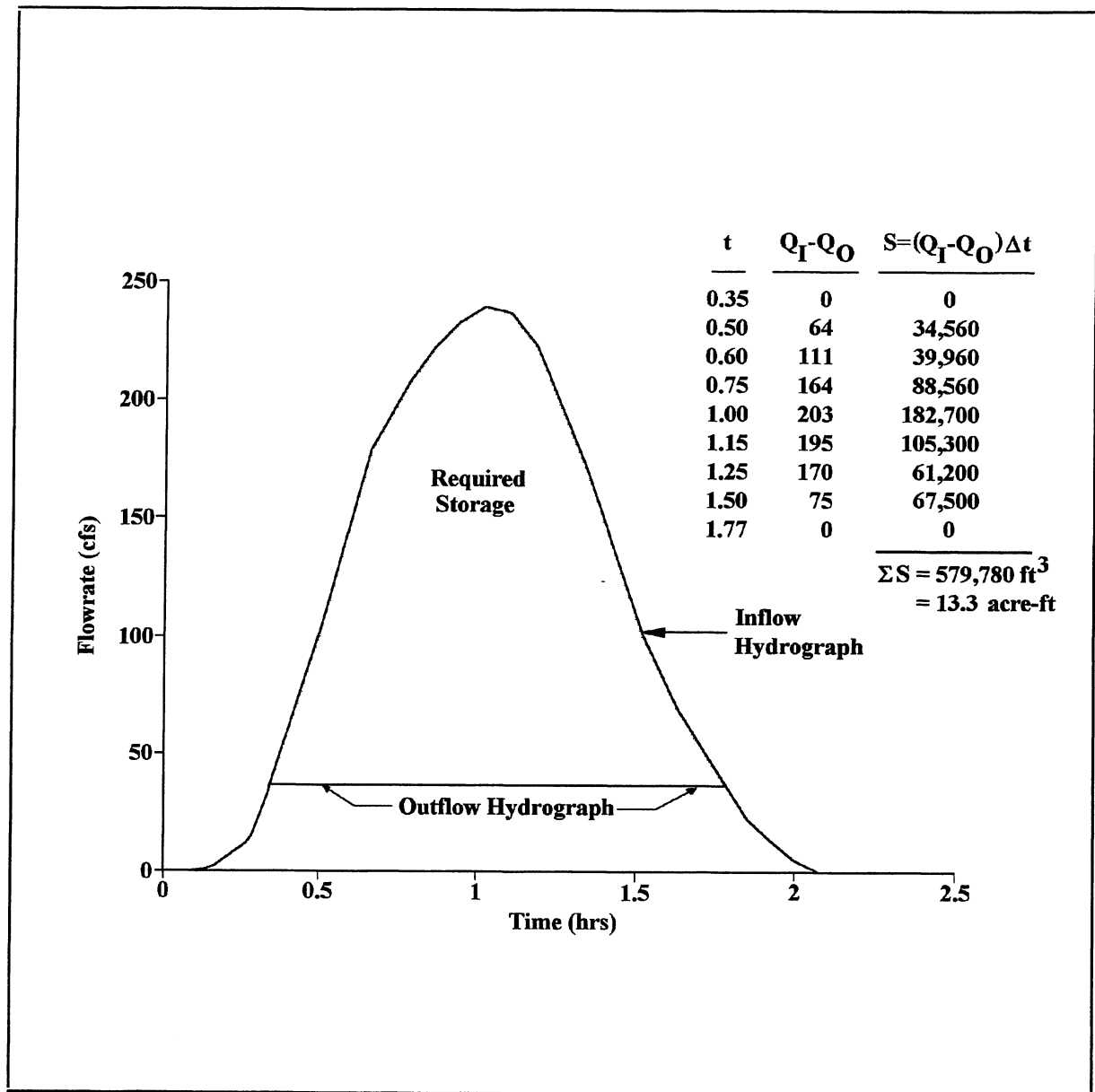


Figure 6.2.8
Storage Determination for Example 6.2.2

6.3 - DESIGN OF STORAGE FACILITIES

General guidelines for the design of storage facilities are outlined in this section. Commonly used design criteria are presented for each type of facility. In the last part of this section the types of outlet structures used for these facilities and applicable equations are discussed. Application of the material in this section is illustrated by Example 6.3.1.

6.3.1 - Retention Ponds

Retention ponds should be constructed for both aesthetic and practical use. Investigation of potential water quality problems associated with low flows in dry weather, direct pollution from surface runoff or high nutrient levels is necessary. High nutrient levels may cause growth of algae and subsequent eutrophication. If any of these problems arise with a retention pond, a detention pond should be considered instead.

The following is a list of some design considerations for retention ponds:

- When possible, retention ponds should be stocked with fish. To provide a habitat suitable for aquatic life, it is recommended that the pond has a surface area of at least one-half acre and a minimum depth of at least 10 feet over at least 25% of the total area. The average depth of the remaining area should be at least 5 feet. This is needed to inhibit insect breeding and weed growth although periodic maintenance may still be necessary to completely control weed growth. A minimum side slope of 3:1 is also recommended. A five foot safety ledge should be constructed at a depth of no more than 18 inches.
- It is recommended that the design include a means of keeping a design water level during periods of prolonged dry weather as a way of enhancing water quality. One method which has been suggested to augment the pond volume and solve these problems is the placement of a fire hydrant near the pond site.
- An emergency outflow spillway or weir should be installed to ensure that flows which exceed the design capacity of the pond do not damage surrounding property.
- Fencing may be required to ensure the safety of children.
- All inlet openings larger than 6 inches should have bars or screens on them to keep children and large animals from entering them and to collect debris.
- Retention facilities can be developed to provide wetland communities rather than being deep holes. The grading can be accomplished to establish sedge meadow and erect emergent wetlands.

6.3.2 - Detention Ponds

Detention ponds should be designed for multiple purposes. In some parts of the country they are used as athletic fields or parks during dry weather. Regardless of its dry weather use, there are certain guidelines which must be followed to ensure that the pond is properly maintained so that it does not become a detriment or hazard.

- The pond should have no side slopes which are greater than 3 feet horizontal to 1 foot vertical. A five foot safety ledge should be constructed at a depth of no more than 18 inches.
- An underdrain system should be constructed to minimize the wetness of the pond bottom. This is to ensure that the bottom does not harbor insects or prohibit recreational dry weather use. This underdrain system may discharge directly into the receiving stream or into a wet well where it is then pumped out to the receiving stream.
- An alternative to the underdrain system is sloping of the pond bottom with at least a 2% grade from inlet to outlet. Another means of eliminating the wetness is to slope the pond to a lined channel which flows through the pond. This gutter may also be used to convey dry weather flow.
- The pond bottom and side slopes should be finished with at least four inches of topsoil and seeded or sodded. This seed or sod should be capable of withstanding periodic flooding conditions.
- Outlets which discharge overland should be provided with rip-rap protection to prohibit erosion.
- An overflow spillway or weir should be placed at the high water elevation and a minimum of 6 inches of freeboard should be provided.
- Screens or bars should be placed over all inlets and outlets greater than 6 inches to protect children and large animals, and to collect debris.
- To maximize water quality benefits the pond inlet and outlet should be located on the opposite ends to provide the maximum settling time within the basin to encourage settling of suspended solids.

6.3.3 - Parking Lot Storage

Parking lots may be utilized for temporary storage of stormwater in two forms. The first and most widely used method is to store runoff in a depressed area. The accumulated runoff is allowed to accumulate and then released through a drain using restrictors. The restrictors may be small diameter pipes, orifices, or small grates. This type of storage requires special design criteria which are presented below.

- The storage areas of the parking lots should be restricted to remote locations or those which cause the least inconvenience to users.
- The maximum depth of water should not exceed 12 inches.
- The parking lot should be drained in at most 30 minutes after the end of rainfall.
- Frequent maintenance should be provided to ensure that the drain openings do not plug up.
- Orifices less than four inches should not be used so that clogging is avoided.
- Emergency overland flow routes shall be established for flood events which fill the storage volume. The elevation of these routes must be lower than the elevation of the surrounding buildings.

Another way of using parking lots as a detention storage facility is by conveying the runoff from paved surfaces to grassed areas or seepage pits located adjacent to lot. Criteria used in determining infiltration rates are discussed in Section 6.3.5.

6.3.4 - Rooftop Storage

The temporary storage of rainfall on rooftops can be an economical approach to temporarily store stormwater. Most current building codes in the northern states require that rooftops withstand live loadings of 30 to 40 pounds per square foot. This is equivalent to about 6 inches of water which is usually much greater than that required for normal detention on flat roofs.

There are obviously several problems associated with rooftop storage. Leakage, overflows, and structural damage are just a few of the potential problems. To minimize these hazards and design the best facility, the following guidelines are recommended.

- Provide extra membrane liners to create a watertight seal on the roof.
- Place overflow drains at the design depth to reduce damage due to clogging of drains.
- Design the roof to structurally withstand a live load of at least 35 pounds per square foot.
- Design the roof so that the maximum depth with a flat roof is 3 inches and the maximum depth with a sloped roof is 6 inches.
- Space each drain to handle a maximum of 10,000 square feet of roof area.
- Determine the allowable outflow rate from the rooftop and then use Table 6.3.1 to size the leader which is the pipe which takes the water from the roof vertically to storm drain, or the horizontal piping to release the water. Check that the depth does not exceed the 3 inches for a flat roof or 6 inches for the sloped roof.

Table 6.3.1
System Sizing Data (Blendermann, 1979)

<i>Pipe Diameter in Inches</i>	<i>Roof Drains and Vertical Leaders (gpm)</i>	<i>Horizontal Storm-Drainage Piping Slope (in /ft)</i>		
		1/8	1/4	1/2
		<i>Flow Capacity (gpm)</i>		
2	30	-	-	-
2.5	54	-	-	-
3	92	34	48	69
4	192	78	110	157
5	360	139	197	278
6	563	223	315	446
8	1208	479	679	958
10	-	863	1217	1725
12	-	1388	1958	2775
15	-	2479	3500	4958

6 3 5 - Infiltration of Stormwater

The temporary storage and subsequent infiltration of stormwater into soil may be accomplished through the use of a basin, trench, or porous asphalt. These are normally used when the receiving stream cannot accept additional runoff, where it is necessary to recharge the groundwater, or where a positive gravity outlet is not feasible. The rate at which the water can percolate through the soil depends upon the soil makeup and the location of the ground water table.

The soil permeability is a measure of the ability of the soil to allow infiltration. Typical values of the coefficient of permeability, denoted by K, are given in Table 6.3.2. This coefficient can be used with Darcy's Law as presented in Equation 6.3.1,

Table 6.3.2
Coefficients of Permeability (Hydraulic Conductivity)

Typical Soil	Permeability K (ft/sec)	Relative Permeability
Coarse, Gravel	over 5.0×10^{-1}	Very Permeable
Sand, Fine Sand	$5.0 \times 10^{-1} - 5.0 \times 10^{-3}$	Medium Permeability
Silty Sand, Dirty Sand	$5.0 \times 10^{-3} - 5.0 \times 10^{-5}$	Low Permeability
Silt	$5.0 \times 10^{-5} - 5.0 \times 10^{-7}$	Very Low Permeability
Clay	less than 5.0×10^{-7}	Practically Impervious

$$Q' = A'Kh_L \quad (6.3.1)$$

where Q' is the flow rate (cfs), A' is the cross-sectional area of soil through which the water flows (ft^2), K is the coefficient of permeability (Table 6.3.2)(ft/sec), and h_L is the headloss or the gradient over a flow distance L (it is recommended that a factor of safety of two be used, i.e. divide Q' in half)(ft/ft).

Equation 6.3.1, which is applicable for a fully saturated soil, is used to determine the required size of the ditch or basin. Figure 6.3.1 shows a typical recharge trench. The void space in the rock fill must have sufficient capacity to detain the difference between the incoming runoff, the water which is infiltrating during the entire storm, and the amount which may be released.

The procedure which is used in determining the storage requirement is outlined below:

- Step 1: Calculate the volume of accumulated runoff for a given time increment, t_d , for the storm duration using the rational method, SCS curve number method, or hydrograph method. $(Q \times t_d)$ (cubic feet).
- Step 2: Calculate the allowable release volume which does not have to be infiltrated, if any. $(Q \times t_d)$.
- Step 3: Calculate the volume of the water flowing through the soil at time t_d . $(Q' \times t_d)$. Q' is found from Equation 6.3.1.
- Step 4: The difference between the inflow volume and the allowable release rate and percolated volume is the required storage.
- Step 5: The largest required storage when all the durations are considered is the design requirement.
- Step 6: Design the basin or trench to have a void volume equal to the maximum required storage. Information concerning the required size of the rock fill and corresponding void ratios may be found in the Design of Small Dams, USBR, 1987.

Porous asphalt may also be used in detaining the water and allowing it to percolate through the soil. This requires a special type of pavement: open graded permeable material (OGPM) with a gravel subbase. There are some installations using this material but most are in experimental stages. Questions about the rigidity, and effects on water quality still remain.

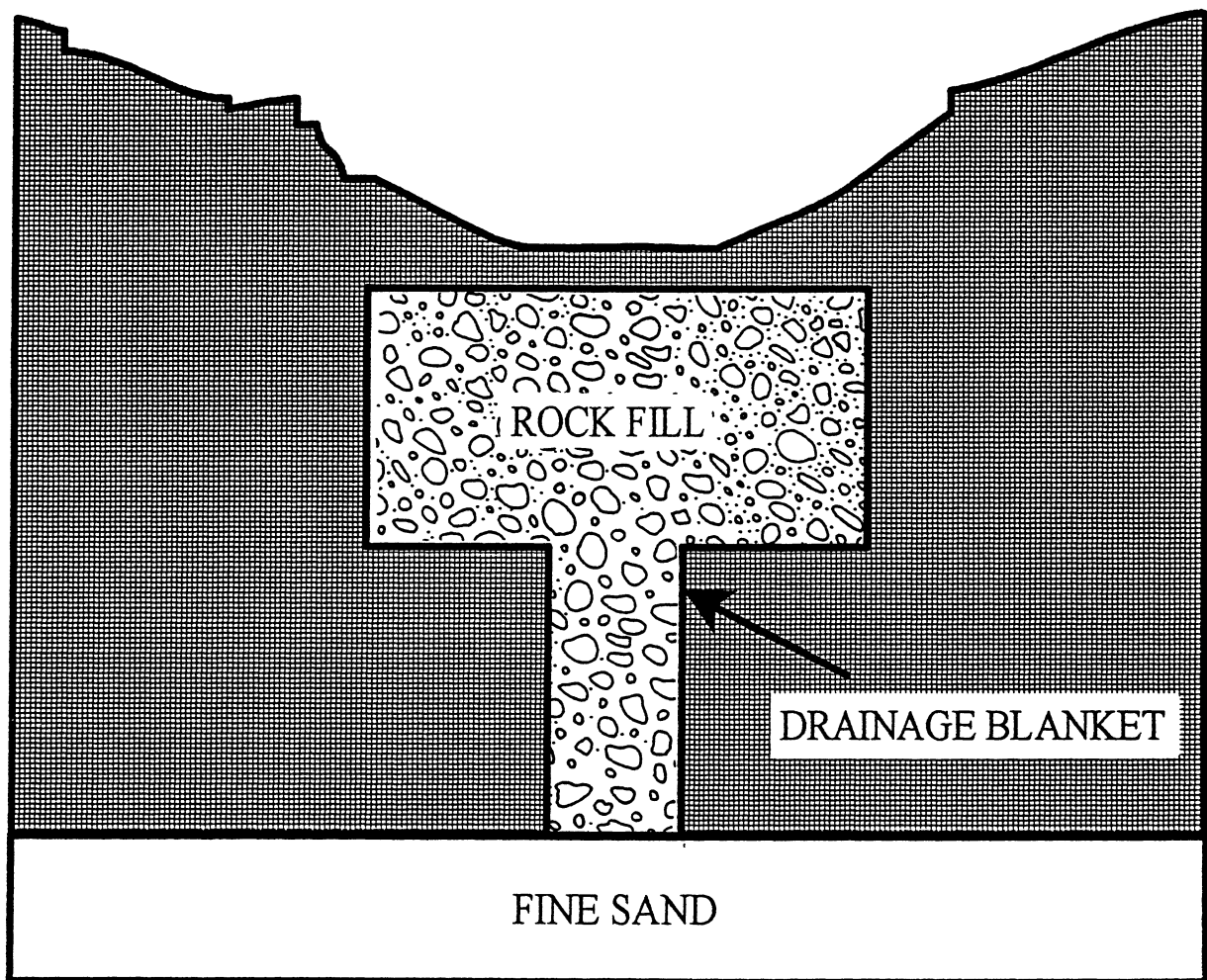


Figure 6.3.1
Design of On-Site Discharge Ditch (Karaca, 1980)
(The pervious drainage blanket shown above can be constructed of
"Bidim" (by Monsanto) or equivalent).

6.3.6 - Outflow Control Devices

The design of any detention facility requires that the outflow be regulated to a maximum flowrate. This is usually accomplished by using standard calibrated devices. The three most commonly used devices are the orifice, weir, and pipe. The following is a description of each of the above and the governing equations.

ORIFICE An orifice is a circular or rectangular opening of a prescribed shape and perimeter through which water flows. The flowrate depends upon the height of water, h_o , above the opening, and the type of orifice. The equation which is used to calculate the flow rate into an orifice is given below.

$$Q = C_d A_o \sqrt{2gh_o} \quad (6.3.2)$$

where C_d is the discharge coefficient, A_o is the area of the orifice (ft^2), g is the acceleration due to gravity (32.2 ft/sec^2), and h_o is the height of water above the center line of the orifice (ft)

Figure 6.3.2 is a typical circular orifice with the above variable defined and Figure 6.3.3 gives discharge coefficients for various types of orifices.

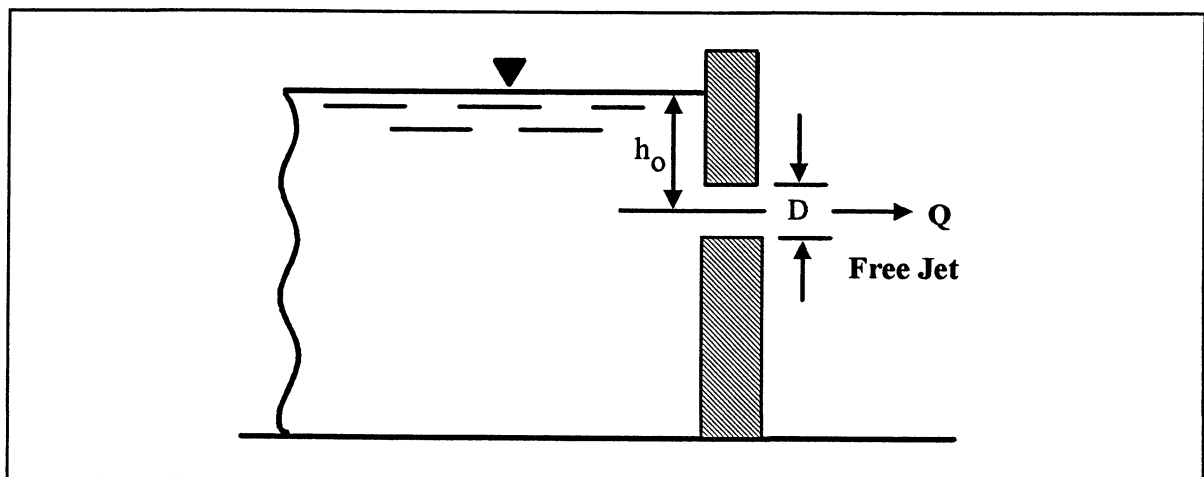
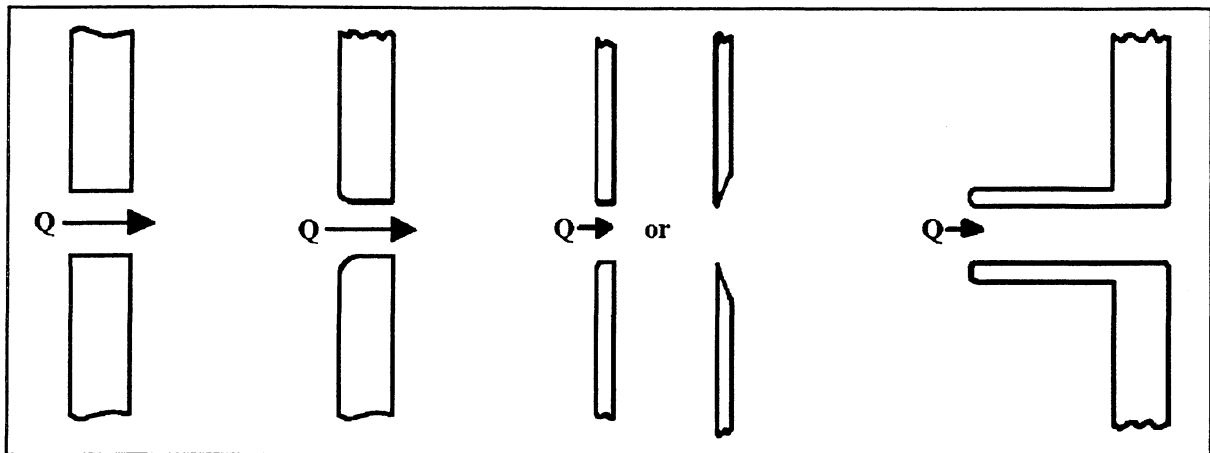


Figure 6.3.2
Typical Orifice



Square-edged
 $C_d = 0.79-0.82$

Round-edged
 $C_d = 0.92-0.98$

Sharp-edged
 $C_d = 0.58-0.64$

Projecting Sharp-edge
 $C_d = 0.50$

Figure 6.3.3
 Discharge Coefficients C_d for Various Types of Orifices (Kuhl, 1977)

WEIR A weir is a depression or cutout in a wall or channel through which water flows. Weirs are often used for controlling overflows. Two types of sharp-crested weirs, rectangular and triangular are discussed below. A typical rectangular weir is shown in Figure 6.3.4. The equation for flow through a rectangular weir is

$$Q = \frac{2}{3} C_d \sqrt{2g} L h_w^{3/2} \quad (6.3.3)$$

where L is the length of the weir in feet, C_d is the discharge coefficient found by experiment or furnished by manufacturer, h_w is the height of the water surface above the weir opening in feet, g is the acceleration due to gravity (32.2 ft/sec^2)

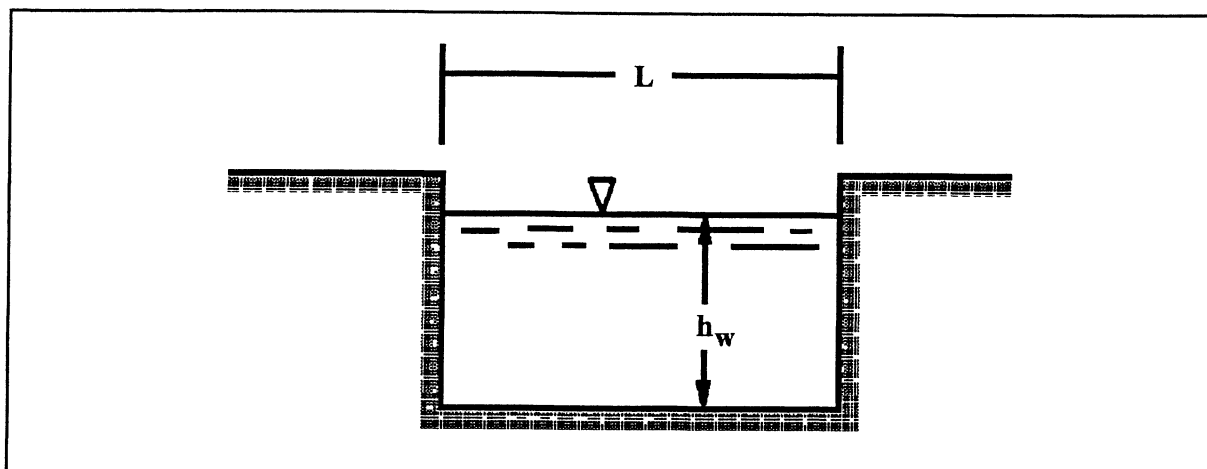


Figure 6 3 4
Typical Rectangular Weir

For a triangular weir shown in Figure 6 3 5, Equation 6.3.4 is used to calculate the flowrate,

$$Q = \frac{8}{15} C_d \sqrt{2g} \tan\theta h_w^{5/2} \quad (6.3.4)$$

where θ is 1/2 the total weir angle and C_d , h_w , and g are the same as in Equation 6.3.3.

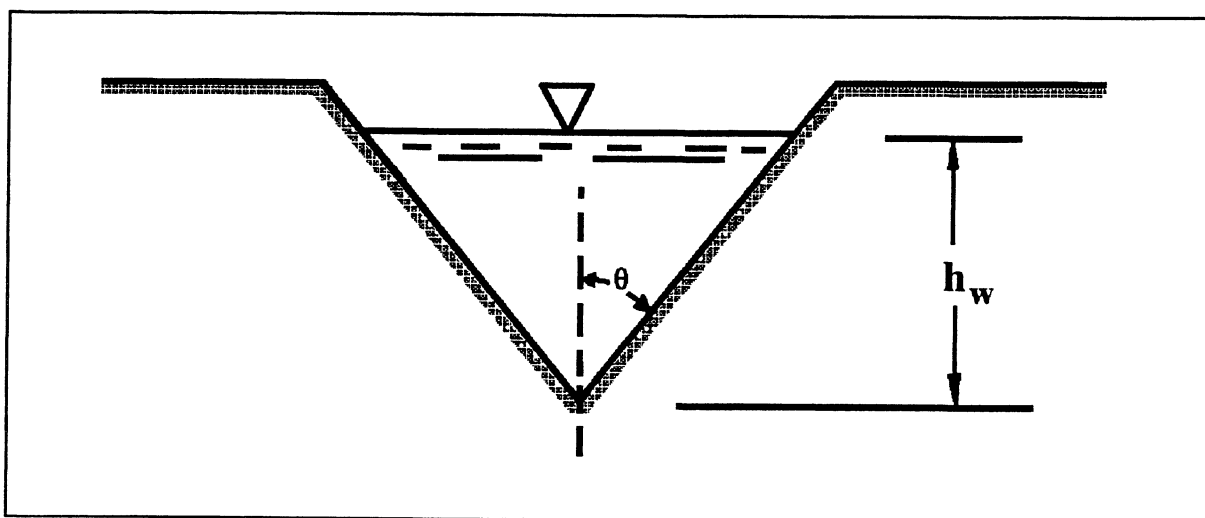


Figure 6 3 5
Typical Triangular Weir

PIPE FLOW: For a pipe flowing full, but not under pressure, Manning's Equation (Equation 4.2.1) may be used to calculate the flowrate. As soon as a depth of water develops above the pipe the flowrate must be calculated using a modified form of the Manning equation and must include entrance and exit losses. Equation 6.3.5 is used to calculate the flowrate under these conditions,

$$Q = A_p \left[\frac{h_p}{\frac{K_e + K_o}{2g} + \frac{2.48 n^2 L}{D^{4/3}}} \right]^{1/2} \quad (6.3.5)$$

where Q is the flowrate in cfs, A_p is the area of the pipe (ft²), K_e is the entrance loss coefficient (given in Table 6.3.3), K_o is the outlet loss coefficient (usually taken as 1.0), D is the pipe diameter (ft), n is Manning's roughness coefficient (Table 4.2.1), L is the length of pipe (ft) (Equation assumes a free jet @ exit), and h_p is the height of water surface above center of pipe opening (ft)

All of the equations presented may be used to calculate the flowrate Q . Often the flowrate is known and the orifice opening, weir size or pipe diameter are the parameters actually required. In these cases the equations are rearranged to solve for the unknown variable. When the computed result does not correspond to a commercially available pipe size, the type of opening, discharge coefficient, or pipe roughness are varied to obtain a readily available pipe of proper size.

Table 6.3.3
Value of K_e (Entrance Losses)
(Bureau of Reclamation, 1987)

<i>Entrance Condition</i>	<i>K_e range</i>	<i>Average</i>
Square-edged inlets installed flush with vertical headwalls	0.43 to 0.70	0.50
Rounded Inlets installed flush with vertical headwalls, $r/D \leq 0.15$	0.08 to 0.27	0.10
Grooved or socket-ended concrete pipe installed flush with vertical headwall	0.10 to 0.33	0.15
Projecting concrete pipe with grooved or socket ends	-	0.20
Projecting steel or corrugated metal pipes	0.5 to 0.9	0.85

Example 6.3.1 (After MWRDGC)

For the developed watershed shown in Figure 6.3.6, calculate the required storage on the rooftop, parking lot and retention/detention pond using the rational method and MWRDGC criteria and size their respective outlet structures. The time of concentration for the undeveloped state is 27 minutes. Use Lafayette, Indiana rainfall data from Example 6.2.1. Use an orifice for the detention pond and a concrete pipe with a length of 100 ft. with a square edged entrance for the parking lot. Determine the size of the pipe for the roof drain using Table 6.3.1.

Solution

For a time of concentration of 27 minutes, the rainfall intensity for a 3-year return period in Lafayette, Indiana is 2.68 inches/hr. Therefore the allowable outflow rate is:

$$O(t) = C_u i_u A_u = (0.15) (2.68) (650 \times 650) \frac{1}{43,560} = 3.90 \text{ cfs}$$

Note that 0.15 is the maximum allowable runoff coefficient

Calculate the developed runoff coefficient, C_D

Type of Cover	Area Calculations	Area (ft ²)	Area (acres)	C	$C_n A_n$
Pavement	150'x150' + 200'x650'	152,500	3.50	0.95	3.33
Roofs	300' x 200'	60,000	1.38	0.95	1.31
Pond	100' x 50'	5,000	0.11	1.0	0.11
Grass	9.7 acres - (3.50 + 1.38 + 0.11)		4.71	0.25	1.18
Total			9.7		5.93

$$C_D = \frac{\sum C_n A_n}{A_T} = \frac{5.93}{9.7} = 0.61$$

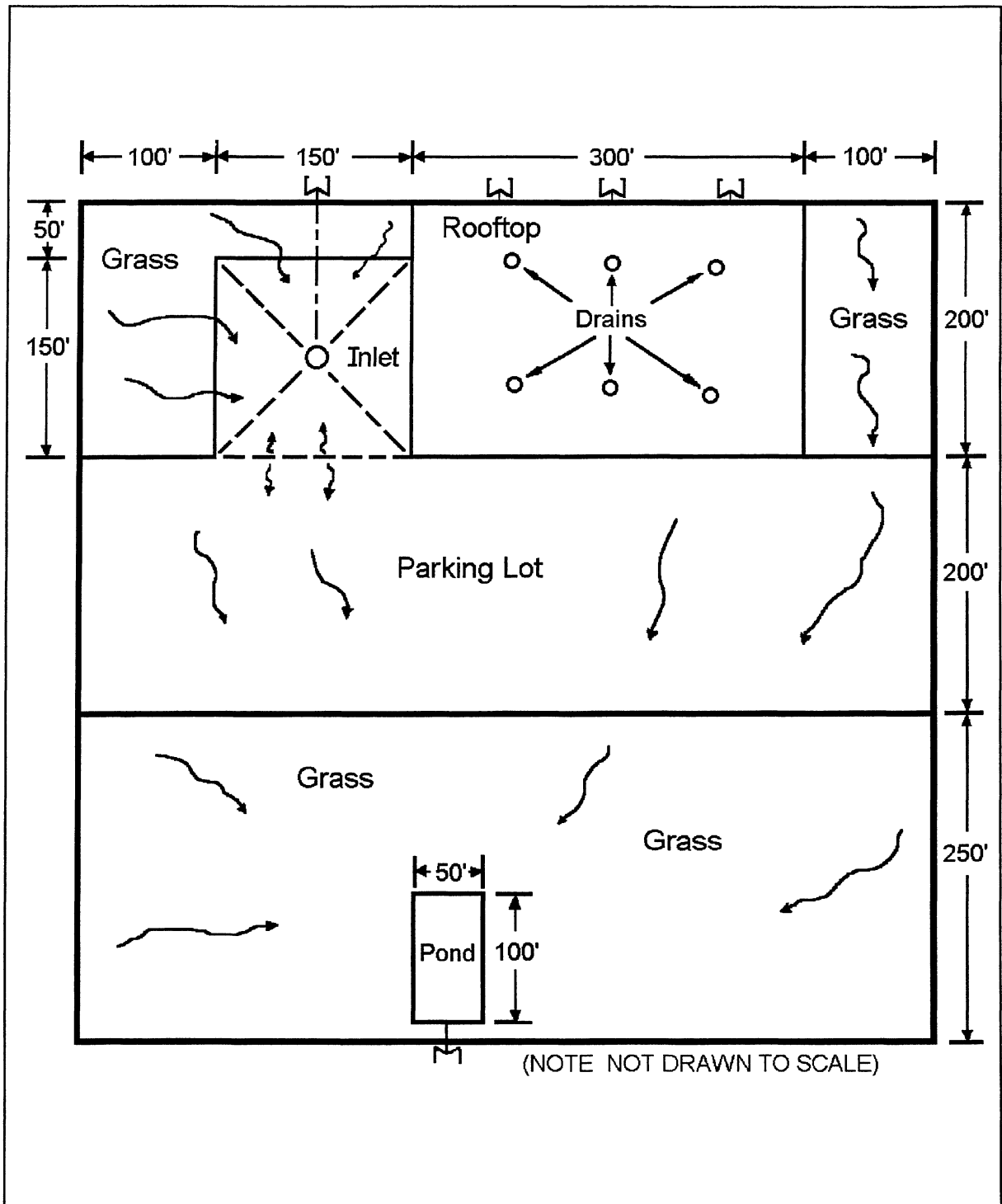


Figure 6.3.6
Hypothetical Watershed for Example Problem 6.3.1 (After MWRDGC)

Project Example 6.3.1 Detention Facility Design Return Period 100 yrs.

Designer TTB Release Rate Return Period 3 yrs.

Watershed Area 9.7 acres

Time of Concentration (undeveloped watershed) 27 minutes

Rainfall Intensity (i_U) 2.68 inches/hr

Undeveloped Runoff Coefficient (C_U) 0.15

Undeveloped Runoff Rate ($O = C_U i_U A_U$) 3.90 cfs

Developed Runoff Coefficient (C_D) 0.61

Storm Duration t_d (hrs)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $I(t_d)$ $(C_D i_d A_D)$ (cfs)	Outflow Rate O $(C_U i_U A_U)$ (cfs)	Storage Rate $I(t_d) - O$ (cfs)	Required Storage $[I(t_d) - O] \frac{t_d}{12}$ (acre-ft)
0.17	7.79	46.09	3.90	42.19	0.59
0.33	5.62	33.25	3.90	29.35	0.82
0.50	4.46	26.39	3.90	22.49	0.94
0.67	3.74	22.13	3.90	18.23	1.01
0.83	3.24	19.17	3.90	15.27	1.06
1.0	2.87	16.98	3.90	13.08	1.09
1.5	2.16	12.78	3.90	8.88	1.11
2.0	1.76	10.41	3.90	6.51	1.09
3.0	1.31	7.75	3.90	3.85	0.96
4.0	1.05	6.21	3.90	2.31	0.77
5.0	0.89	5.27	3.90	1.37	0.57
6.0	0.78	4.62	3.90	0.72	0.36
7.0	0.69	4.08	3.90	0.18	0.11
8.0	0.62	3.67	3.90	*	*
9.0	0.57	3.37	3.90	*	*
10.0	0.53	3.14	3.90	*	*

Peak
Storage

* Since $I(t_d) - O < \text{zero}$, there is no storage needed.

Figure 6.3.7
Detention Storage Calculations for the Entire Basin Using the Rational Method

Using the outflow rate and the developed runoff coefficient, the storage volume for the entire basin is calculated by using the rational method and Figure 6.2.2. From the calculations shown by Figure 6.3.7, the maximum required storage is 1.11 acre-feet. This is the sum of the rooftop, parking lot and pond storage. Consequently, each facility must be designed separately so that the total release rate is less than or equal to 3.90 cfs and the total storage is 1.11 acre-feet

The designer must decide how much runoff may be released from each facility. For this example, the rooftop will release 1.40 cfs, the pond 2.0 cfs and the parking lot 0.50 cfs. In many cases a trial and error procedure is necessary to ensure that each facility does not exceed its maximum storage capacity.

Rooftop

The total area of rooftop is 1.38 acres and the developed runoff coefficient is 0.95. Using this information, the storage can be computed as shown in Figure 6.3.9. The peak storage is found to be 0.20 acre-feet. This volume will be detained in the "cells" shown below in Figure 6.3.8, based on the 10,000 ft²/drain criteria outlined in Section 6.3.4. Knowing the volume, the depth can be computed as follows.

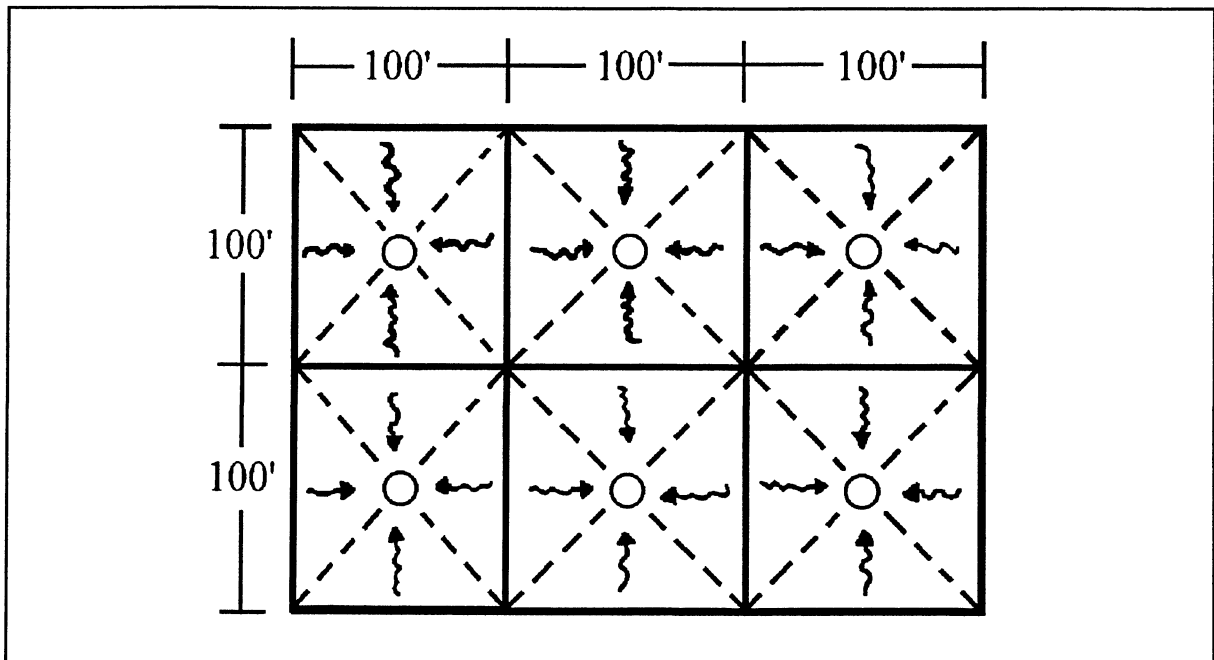


Figure 6.3.8
Roof Drain Configuration

Project Example 6.3.1 Detention Facility Design Return Period 100 yrs.

Designer TTB Release Rate Return Period 3 yrs.

Watershed Area 1.38 acres

Time of Concentration (undeveloped watershed) -- minutes

Rainfall Intensity (i_U) -- inches/hr

Undeveloped Runoff Coefficient (C_U) --

Undeveloped Runoff Rate ($O = C_U i_U A_U$) 0.50 cfs

Developed Runoff Coefficient (C_D) 0.95

Storm Duration t_d (hrs)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $I(t_d)$ ($C_D i_d A_D$) (cfs)	Outflow Rate O ($C_U i_U A_U$) (cfs)	Storage Rate $I(t_d) - O$ (cfs)	Required Storage $\left[I(t_d) - O \right] \frac{t_d}{12}$ (acre-ft)
0.17	7.79	10.21	1.40	8.81	0.12
0.33	5.62	7.37	1.40	5.97	0.17
0.50	4.46	5.85	1.40	4.45	0.19
0.67	3.74	4.90	1.40	3.50	0.19
0.83	3.24	4.25	1.40	2.85	0.20
1.0	2.87	3.76	1.40	2.36	0.20
1.5	2.16	2.83	1.40	1.43	0.18
2.0	1.76	2.31	1.40	0.91	0.15
3.0	1.31	1.72	1.40	0.32	0.08
4.0	1.05	1.38	1.40	*	*
5.0	0.89	1.17	1.40	*	*
6.0	0.78	1.02	1.40	*	*
7.0	0.69	0.90	1.40	*	*
8.0	0.62	0.81	1.40	*	*
9.0	0.57	0.75	1.40	*	*
10.0	0.53	0.69	1.40	*	*

Peak
Storage

* Since $I(t_d) - O < \text{zero}$, there is no storage needed.

Figure 6.3.9
Detention Storage Calculations for Rooftop Using the Rational Method

The volume of each cell is

$$V = \frac{1}{3}Ad \therefore \text{solving for } d: \quad d = \frac{3V}{A}$$

Since there are six "cells", the maximum depth required for each is:

$$d = \frac{3(0.20 \text{ acre-ft})}{1.38 \text{ acres}} = 0.43 \text{ ft}$$

In summary, there are six drains needed. Each one must be able to handle a five-inch head and release

$$\frac{1.40 \text{ cfs}}{6} = 0.233 \text{ cfs} = 104.57 \text{ gpm}$$

From Table 6 3 1, it is found that a four-inch leader is required

Parking Lot

The total area which contributes to the parking lot inlet is 1.15 acres. The developed runoff coefficient is found below:

Type of Cover	Area Calculations	Area (ft ²)	Area (acres)	C	C _n A _n
Pavement	150'x150'	22,500	0.52	0.95	0.49
Grass	100'x200' + 150'x50'	27,500	0.63	0.25	0.16
Total			1.15		0.65

$$C_D = \frac{\Sigma C_n A_n}{A_T} = \frac{0.65}{1.15} = 0.57$$

With the allowable release of 0.50 cfs, the storage can be calculated as shown in Figure 6.3.11 where the peak storage is found to be 0.12 acre-feet. The figure below shows the parking lot layout. The depth in the parking lot is.

$$d = \frac{3V}{A} = \frac{3(0.12 \text{ acre-ft})}{0.52} = 0.70 \text{ ft}$$

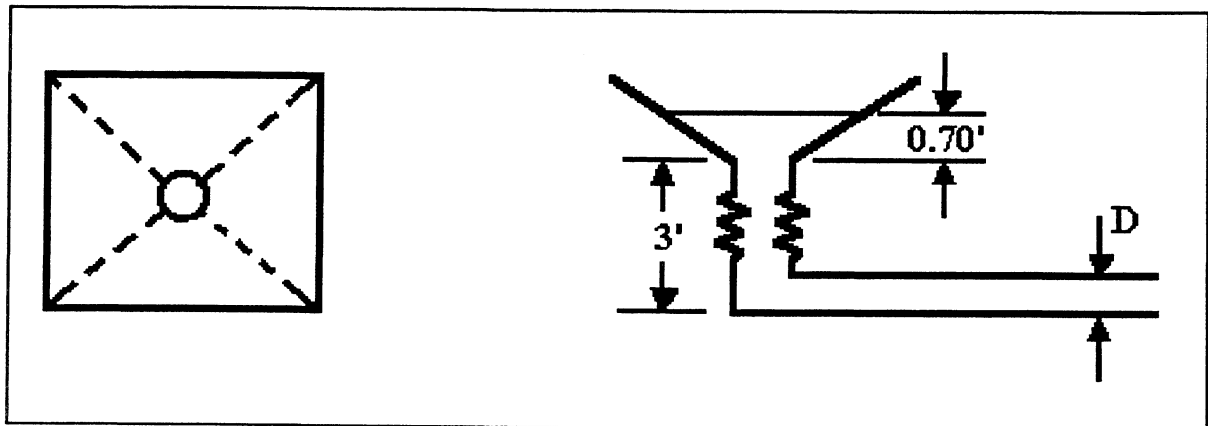


Figure 6.3.10
Drainage for Parking Lot Segment

Project Example 6.3.1 Detention Facility Design Return Period 100 yrs.

Designer TTB Release Rate Return Period 3 yrs.

Watershed Area 1.15 acres

Time of Concentration (undeveloped watershed) -- minutes

Rainfall Intensity (i_U) -- inches/hr

Undeveloped Runoff Coefficient (C_U) --

Undeveloped Runoff Rate ($O = C_U i_U A_U$) 0.50 cfs

Developed Runoff Coefficient (C_D) 0.57

Storm Duration t_d (hrs)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $I(t_d)$ $(C_D i_d A_D)$ (cfs)	Outflow Rate $O(t_d)$ $(C_U i_U A_U)$	Storage Rate $I(t_d) - O(t_d)$ (cfs)	Required Storage $[I(t_d) - O(t_d)] \frac{t_d}{12}$ (acre-ft)
0.17	7.79	5.11	0.50	4.61	0.06
0.33	5.62	3.68	0.50	3.18	0.09
0.50	4.46	2.92	0.50	2.42	0.10
0.67	3.74	2.45	0.50	1.95	0.11
0.83	3.24	2.12	0.50	1.62	0.11
1.0	2.87	1.88	0.50	1.38	0.12
1.5	2.16	1.42	0.50	0.92	0.11
2.0	1.76	1.15	0.50	0.65	0.11
3.0	1.31	0.86	0.50	0.36	0.09
4.0	1.05	0.69	0.50	0.19	0.06
5.0	0.89	0.58	0.50	0.08	0.03
6.0	0.78	0.51	0.50	0.01	0.01
7.0	0.69	0.45	0.50	*	*
8.0	0.62	0.41	0.50	*	*
9.0	0.57	0.37	0.50	*	*
10.0	0.53	0.35	0.50	*	*

Peak
Storage

* Since $I(t_d) - O < \text{zero}$, there is no storage needed.

Figure 6.3.11
Detention Storage Calculations for the Parking Lot Using the Rational Method

If a 3-foot deep inlet is used, the total head to the centerline of the pipe is

$$(0.70 + 3.0 - \frac{D}{2})$$

The required pipe size is determined using Equation 6.3.5 and an iterative process:
(take $n = 0.013$)

$$0.5 = A_p \left[\frac{h_p}{\frac{K_e + K_o}{2g} + \frac{2.87 n^2 L}{D^{4/3}}} \right]^{1/2} = \frac{\pi D^2}{4} \left[\frac{3.7 - \frac{D}{2}}{\frac{1.43}{64.4} + \frac{2.87 (0.013)^2 100}{D^{4/3}}} \right]^{1/2}$$

$$D = 0.39 \text{ feet} = 4.5 \text{ inches}$$

Pond

The total area of the basin which contributes to the pond is 7.17 acres. The runoff coefficient calculations are shown below:

Type of Cover	Area Calculations	Area (ft ²)	Area (acres)	C	C _r A _n
Parking Lot	200' x 650'	130,000	2.98	0.95	2.83
Pond	100' x 50'	5,000	0.11	1.0	0.11
Grass	100'x200' + 250'x650' -0.11 acres		4.08	0.25	1.02
Total			7.17		3.96

$$C_D = \frac{\sum C_r A_n}{A_T} = \frac{3.96}{7.17} = 0.55$$

Using the undeveloped runoff coefficient and an outflow rate of 2 cfs, the storage volumes can be calculated as shown in Figure 6.3.12. The required storage volume is 0.82 acre-ft. Since the permanent area is 5,000 ft², the volume required is determined by using a minimum 3:1 slope.

If a depth of 4.75 feet is used, the perimeter of the pond will be 78.5 x 128.5 and the volume will be 0.82 acre-feet as shown below:

$$V = \left[\frac{(100 + 6(4.75)) * (50 + 6(4.75)) + (100)(50)}{2} \right] 4.75 = 0.82 \text{ ac-ft}$$

Project Example 6.3.1 Detention Facility Design Return Period 100 yrs.

Designer TTB Release Rate Return Period 3 yrs.

Watershed Area 7.17 acres

Time of Concentration (undeveloped watershed) -- minutes

Rainfall Intensity (i_U) -- inches/hr

Undeveloped Runoff Coefficient (C_U) --

Undeveloped Runoff Rate ($O = C_U i_U A_U$) 2.0 cfs

Developed Runoff Coefficient (C_D) 0.55

Storm Duration t_d (hrs)	Rainfall Intensity i_d (inches/hr)	Inflow Rate $I(t_d)$ $(C_D i_d A_D)$ (cfs)	Outflow Rate $O(t_d)$ $(C_U i_U A_U)$ (cfs)	Storage Rate $I(t_d) - O(t_d)$ (cfs)	Required Storage $[I(t_d) - O(t_d)] \frac{t_d}{12}$ (acre-ft)
0.17	7.79	30.72	2.0	28.72	0.40
0.33	5.62	22.16	2.0	20.16	0.56
0.50	4.46	17.59	2.0	15.59	0.65
0.67	3.74	14.75	2.0	12.75	0.71
0.83	3.24	12.78	2.0	10.78	0.75
1.0	2.87	11.32	2.0	9.32	0.78
1.5	2.16	8.52	2.0	6.52	0.81
2.0	1.76	6.94	2.0	4.94	0.82
3.0	1.31	5.17	2.0	3.17	0.79
4.0	1.05	4.14	2.0	2.14	0.71
5.0	0.89	3.51	2.0	1.51	0.63
6.0	0.78	3.08	2.0	1.08	0.54
7.0	0.69	2.72	2.0	0.72	0.42
8.0	0.62	2.44	2.0	0.44	0.30
9.0	0.57	2.25	2.0	0.25	0.19
10.0	0.53	2.09	2.0	0.09	0.08

Peak
Storage

* Since $I(t_d) - O < \text{zero}$, there is no storage needed.

Figure 6.3.12
Detention Storage Calculations for the Detention Pond Using the Rational Method

Conclusion:

<i>Type of Cover</i>	<i>Storage (acre-feet)</i>	<i>Release Rate (cfs)</i>
Pond	0.82	2.00
Rooftop	0.20	1.40
Parking Lot	0.12	0.50
Total	1.14	3.90

Example 6.3.2 (TR-55 Methodology)

Using the data from Example 6.2.1, compute the detention storage requirement using the Quick TR-55 computer program. Use AMC II, 24-hour 100-year rainfall depth, time of concentration of 20 minutes, and a SCS Type II distribution.

In summary:

- CN = 75.9
- t_c = 20 minutes
- A = 10 acres
- O = 5.27 cfs

The rainfall depth can be found with the information from Example 6.2.1 for Lafayette, Indiana:

$$a' = a_1 i_1^{10} \log(10^{(2-X)} T_r^{(X-1)}) = (28.6)(1.95) \log(10^{0.6} 100^{0.4}) = 78.08$$

$$i_{24}^{100} = \frac{78.08}{(60(24) + 9.2)^{0.78}} = 0.267 \text{ inches/hr}$$

With an intensity of 0.267 inches/hr the corresponding depth is $(24 \times 0.267) = 6.41$ inches.

Using Quick TR-55, the peak inflow rate is 41 cfs for the site and using the approximate detention storage procedure within the program, the volume is found to be 1.6 acre-feet. The difference between the rational method result of 0.56 acre-feet, from Example 6.2.1, and this example is because the Quick TR-55 computer program is less accurate as the q_o/q_i ratio approaches the design limits. When the storage volume required is small, the shape of the outflow hydrograph is sensitive to the rate of rise of the inflow hydrograph (SCS, 1986).

>>>> GRAPHICAL PEAK DISCHARGE METHOD <<<<

HERPICC EXAMPLE 6.3.2
TR-55 METHODOLOGYCALCULATED
DISK FILE: c:EX63-2 .GPD

Drainage Area	(acres)	10	---	0.0156 sq.mi.
Runoff Curve Number	(CN)	76		
Time of Concentration, Tc	(hrs)	0.275		
Rainfall Distribution	(Type)	II		
Pond and Swamp Areas	(%)	0	---	0.0 acres
			Storm #1	Storm #2
			-----	-----
Frequency (years)		100		
Rainfall, P, 24-hr (in)		6.41		
Initial Abstraction, Ia (in)		0.632	0.632	0.632
Ia/p Ratio		0.099	0.000	0.000
Unit Discharge, * qu (csm/in)		702	0	0
Runoff, Q (in)		3.74	0.00	0.00
Pond & Swamp Adjustment Factor		1.00	1.00	1.00
PEAK DISCHARGE, qp (cfs)		41	0	0

Summary of Computations for qu

Ia/p	#1	0.100	0.000	0.000
C0	#1	2.553	0.000	0.000
C1	#1	-0.615	0.000	0.000
C2	#1	-0.164	0.000	0.000
qu (csm)	#1	702.336	0.000	0.000
Ia/p	#2	0.100	0.000	0.000
C0	#2	2.553	0.000	0.000
C1	#2	-0.615	0.000	0.000
C2	#2	-0.164	0.000	0.000
qu (csm)	#2	702.336	0.000	0.000
* qu (csm)		702	0	0

* Interpolated for computed Ia/p ratio (between Ia/p #1 & Ia/p #2)
If computed Ia/p exceeds Ia/p limits, bounding limit for Ia/p is used.

$$\log(qu) = C0 + (C1 * \log(Tc)) + (C2 * (\log(Tc))^2)$$

$$qp \text{ (cfs)} = qu \text{ (csm)} * \text{Area(sq.mi.)} * Q \text{ (in.)} * (\text{Pond \& Swamp Adj.})$$

Quick TR-55 Version: 5.46 S/N: 1803000004

>>>> DETENTION STORAGE ESTIMATE <<<<

HERPICC EXAMPLE 6.3.2
TR-55 METHODOLOGY
DETENTION STORAGE ESTIMATE

CALCULATED
DISK FILE: c:EX63-2 .DET

Drainage Area	(acres)	10	0.0156 sq.mi.
Rainfall Distribution	(Type)	II	

	Storm #1	Storm #2	Storm #3
	-----	-----	-----
Frequency (years)	100		
Peak Inflow, qi (cfs)	41		
Inflow Runoff, Q (in)	3.74		
Peak Outflow, qo (cfs)	5.27		
qo/qi Ratio	0.129	0.000	0.000
* Vs/Vr Ratio	0.524	0.000	0.000
Inflow Volume, Vr (ac-ft)	3.1	0.0	0.0
STORAGE VOLUME, Vs (ac-ft)	1.6	0.0	0.0

Summary of Volume Computations

C0	0.682	0.682	0.682
C1	-1.430	-1.430	-1.430
C2	1.640	1.640	1.640
C3	-0.804	-0.804	-0.804
* Vs/Vr	0.524	0.000	0.000

$$* \text{ Vs/Vr} = C0 + (C1*(qo/qi)) + (C2*(qo/qi)^2) + (C3*(qo/qi)^3)$$

6.4 - COMPUTER PROGRAM

```

AT
C
C   AUTHORS: DR. D.D. GRAY AND R.B. BOWERS
C   DATE STARTED: DEC. 22, 1986
C   LAST MODIFIED: APRIL 10, 1988
C
C   THIS PROGRAM USES THE MODIFIED RATIONAL METHOD TO
C   SIZE DETENTION STORAGE.
C   REFERENCE: HERPICC COUNTY STORM DRAINAGE MANUAL BY C.B. BURKE
C
C   ND = NUMBER OF DURATIONS CONSIDERED
C   PARAMETER (ND=16)
C   CHARACTER LABEL*80
C   DIMENSION T(ND),RAIN(ND)
C
C   ARRAY T(ND) CONTAINS STORM DURATIONS IN HOURS
C   AS RECOMMENDED BY MSDGC.
C   DATA T/0.17, 0.33, 0.50, 0.67, 0.83, 1.0, 1.5, 2.0, 3.0, 4.0,
C   & 5.0, 6.0, 7.0, 8.0, 9.0, 10.0/
C
C   ARRAY RAIN(ND) CONTAINS INTENSITIES FOR THE CITY
C   AND THE RETURN PERIOD TR.
C   THE FOLLOWING INTENSITIES (7.6,5.5,...0.49) ARE FOR CHICAGO
C   DATA RAIN/7.6, 5.5, 4.4, 3.7, 3.2, 2.8, 2.1, 1.7, 1.2, 1.0,
C   & 0.84, 0.73, 0.65, 0.58, 0.53, 0.49/
C
C   TR IS THE RETURN PERIOD
C   DATA TR/100./
C
C   FOR EACH CASE THERE ARE 2 CARDS OF INPUT DATA.
C
C   CARD 1 (A80):
C   LABEL = DESCRIPTIVE TITLE
C   CARD 2 (6F10.0):
C   TU = UNDEVELOPED RETURN PERIOD (YEARS)
C   RUND = UNDEVELOPED RAINFALL INTENSITY (INCHES/HOUR)
C   CU = UNDEVELOPED RUNOFF COEFFICIENT
C   AU = UNDEVELOPED AREA (ACRES)
C   AD = DEVELOPED AREA (ACRES)
C   CD = DEVELOPED RUNOFF COEFFICIENT
C
1 READ(5,100,END=99) LABEL
100 FORMAT(A80)
   READ(5,101) TU,RUND,CU,AU,AD,CD
101 FORMAT(6F10.0)
   WRITE(6,200) LABEL
200 FORMAT('1',/,1X,A80,/)
   WRITE(6,201) TU,RUND,CU,AU,AD,CD
201 FORMAT(' UNDEVELOPED RETURN PERIOD (YEARS) = ',
&6X,F10.0,/, ' UNDEVELOPED RAINFALL INTENSITY (INCH/HOUR) = ',
&F10.3,/, ' UNDEVELOPED RUNOFF COEFFICIENT = ',12X,F10.3,/,
&' UNDEVELOPED AREA (ACRES) = ',18X,F10.3,/,
&' DEVELOPED AREA (ACRES) = ',20X,F10.3,/,
&' DEVELOPED RUNOFF COEFFICIENT = ',14X,F10.3,/)
C   CALCULATE ALLOWABLE OUTFLOW USING THE RATIONAL METHOD
C   OUT=CU*AU*RUND
C   WRITE(6,202) OUT
202 FORMAT(' ALLOWABLE OUTFLOW RATE (CFS) = ',14X,F10.3,/)
   WRITE(6,203)

```

```

203 FORMAT(' DURATION INTENSITY INFLOW',5X,' OUTFLOW'
&,1X,' STORAGE RATE',1X,' STORAGE',/,2X,' HOURS'
&,5X,' IN/HOUR',5X,' CFS',8X,' CFS',8X,' CFS',6X,' AC-FT',/)
VOL=0.
C
C CALCULATE STORAGE FOR EACH DURATION
C
DO 10 I=1,ND
FLOWIN=CD*AD*RAIN(I)
DELQ=FLOWIN-OUT
STOR=DELQ*T(I)/12.
WRITE(6,204) T(I),RAIN(I),FLOWIN,OUT,DELQ,STOR
204 FORMAT(1F7.2,5F12.2)
IF(STOR.GT.VOL) VOL=STOR
10 CONTINUE
WRITE(6,205) VOL
205 FORMAT(//,' MAXIMUM REQUIRED STORAGE (AC-FT) =',F12.2)
GOTO 1
99 CONTINUE
STOP
END

```

```

TEST CASE FOR MODIFIED RATIONAL METHOD
3.      1.8      0.15      13.4      13.4      0.36
SECOND TEST CASE FOR MODIFIED RATIONAL METHOD
3.      1.8      0.15      13.4      13.4      0.50

```


TEST CASE FOR MODIFIED RATIONAL METHOD

UNDEVELOPED RETURN PERIOD (YEARS) = 3.
 UNDEVELOPED RAINFALL INTENSITY (INCH/HOUR) = 1.800
 UNDEVELOPED RUNOFF COEFFICIENT = 0.150
 UNDEVELOPED AREA (ACRES) = 13.400
 DEVELOPED AREA (ACRES) = 13.400
 DEVELOPED RUNOFF COEFFICIENT = 0.360
 ALLOWABLE OUTFLOW RATE (CFS) = 3.618

DURATION HOURS	INTENSITY IN/HOUR	INFLOW CFS	OUTFLOW CFS	STORAGE RATE CFS	STORAGE AC-FT
0.17	7.60	36.66	3.62	33.04	0.47
0.33	5.50	26.53	3.62	22.91	0.63
0.50	4.40	21.23	3.62	17.61	0.73
0.67	3.70	17.85	3.62	14.23	0.79
0.83	3.20	15.44	3.62	11.82	0.82
1.00	2.80	13.51	3.62	9.89	0.82
1.50	2.10	10.13	3.62	6.51	0.81
2.00	1.70	8.20	3.62	4.58	0.76
3.00	1.20	5.79	3.62	2.17	0.54
4.00	1.00	4.82	3.62	1.21	0.40
5.00	0.84	4.05	3.62	0.43	0.18
6.00	0.73	3.52	3.62	-0.10	-0.05
7.00	0.65	3.14	3.62	-0.48	-0.28
8.00	0.58	2.80	3.62	-0.82	-0.55
9.00	0.53	2.56	3.62	-1.06	-0.80
10.00	0.49	2.36	3.62	-1.25	-1.05

MAXIMUM REQUIRED STORAGE (AC-FT) = 0.82

SECOND TEST CASE FOR MODIFIED RATIONAL METHOD

UNDEVELOPED RETURN PERIOD (YEARS) = 3.
 UNDEVELOPED RAINFALL INTENSITY (INCH/HOUR) = 1.800
 UNDEVELOPED RUNOFF COEFFICIENT = 0.150
 UNDEVELOPED AREA (ACRES) = 13.400
 DEVELOPED AREA (ACRES) = 13.400
 DEVELOPED RUNOFF COEFFICIENT = 0.500
 ALLOWABLE OUTFLOW RATE (CFS) = 3.618

DURATION HOURS	INTENSITY IN/HOUR	INFLOW CFS	OUTFLOW CFS	STORAGE RATE CFS	STORAGE AC-FT
0.17	7.60	50.92	3.62	47.30	0.67
0.33	5.50	36.85	3.62	33.23	0.91
0.50	4.40	29.48	3.62	25.86	1.08
0.67	3.70	24.79	3.62	21.17	1.18
0.83	3.20	21.44	3.62	17.82	1.23
1.00	2.80	18.76	3.62	15.14	1.26
1.50	2.10	14.07	3.62	10.45	1.31
2.00	1.70	11.39	3.62	7.77	1.30
3.00	1.20	8.04	3.62	4.42	1.11
4.00	1.00	6.70	3.62	3.08	1.03
5.00	0.84	5.63	3.62	2.01	0.84
6.00	0.73	4.89	3.62	1.27	0.64
7.00	0.65	4.35	3.62	0.74	0.43
8.00	0.58	3.89	3.62	0.27	0.18
9.00	0.53	3.55	3.62	-0.07	-0.05
10.00	0.49	3.28	3.62	-0.33	-0.28

MAXIMUM REQUIRED STORAGE (AC-FT) = 1.31

CHAPTER 6 - REFERENCES

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Chapter 7

STORM SEWER SYSTEM DESIGN

Chapter 7 - STORM SEWER SYSTEM DESIGN

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Chapter 7 - STORM SEWER SYSTEM DESIGN

LIST OF PARAMETERS

a_i, d', n', X	Coefficients to determine the rainfall intensity by Chen's method
A	Area of watershed under consideration (acres).
C	Ratio of peak runoff rate to average rainfall rate over the time of concentration
d	Diameter of the culvert (ft)
H	Headwater depth (ft)
i	Rainfall intensity (in/hr)
i_t^{Tr}	Rainfall intensity corresponding to time, t, and recurrence interval, T_r
L	Length (ft)
Q	Peak discharge (cfs)
Q_p	Capacity of the pipe flowing full (cfs)
Q_r	Runoff from rational method (cfs)
S	Slope (ft/ft)
t_c	Time of concentration (minutes)
t_t	Travel time (minutes)
V	Velocity (ft/sec)
y_c	Critical depth (ft)
y_t	Tailwater depth (ft)

Chapter 7 - STORM SEWER SYSTEM DESIGN

7.1 - INTRODUCTION

The design of a storm sewer system utilizes all the material contained in the previous five chapters. The rainfall data presented in Chapter 2 is used with the runoff prediction methods in Chapter 3 to size the pipes or channels using Manning's Equation (Chapter 4, Equation 4.2.1). When inlets and gutters are to be included in the design or when storage facilities are mandated or desired, Chapters 5 and 6 may be used. Each of the above mentioned are essential components of the system and must be carefully considered.

This chapter presents an outline of the interactive components and references other parts of the manual for information pertaining to each component. Methods which are commonly employed for the design of storm sewers and the basic principles of culvert design are presented. Design criteria applicable for the state of Indiana are outlined and the various materials used for storm sewers and culverts are listed. Examples at the end of the chapter illustrate applications.

7.2 - METHODS EMPLOYED IN THE SIZING OF STORM SEWERS

In Chapter 3, methods were presented by which the quantity of precipitation which becomes stormwater runoff can be computed. The rational method, Soil Conservation Service Curve Number Method and synthetic hydrograph methods were all discussed. Since the curve number method computes only a volume and the hydrographs require a considerable amount of computation, they are not usually employed in complex storm sewer designs unless computer programs are available. Consequently, the rational method, by virtue of its simplicity and ease of application, is the most frequently used method throughout the world (UNESCO, 1977).

However, because of the shortcomings of the rational method and the rapid development and availability of personal computers, programs have been developed which incorporate, in varying detail, hydraulic-hydrologic principles in an attempt to better simulate the rainfall-runoff process. This section discusses the rational method as employed in storm sewer design and one of several available computer programs for storm sewer design.

Table 7.2.1
Components in Storm Sewer Design

<u>Description</u>	<u>Reference</u>
Pipes	
(a) Materials, types	Section 7.5
(b) Sizing	Section 7.2
(c) Design criteria	Section 7.4
(d) Roughness coefficients	Section 4.2, Table 4.2.1
Culverts	
(a) Materials, types	Section 7.5
(b) Sizing	Section 7.3
(c) Design criteria	Section 7.4
(d) Roughness coefficients	Section 4.2, Table 4.2.1
Curbs and Gutters	
(a) Flow in gutters	Section 5.2
(b) Gutter inlets	Section 5.3.1
(c) Curb inlets	Section 5.3.2
(d) Combined inlets	Section 5.3.3
Storage Facilities	
(a) Retention pond	Section 6.3.1
(b) Detention pond	Section 6.3.2
(c) Parking lot storage	Section 6.3.3
(d) Rooftop storage	Section 6.3.4

7.2.1 - Sizing Storm Sewers with the Rational Method

The theory and assumptions of the rational method were discussed in Chapter 3 (Section 3.2). The method is very easily extended for the design of a storm sewer system and for this reason is widely used.

As presented in Chapter 3, the rational formula is written as:

$$Q = CiA \quad (7.2.1)$$

where Q = peak discharge (cfs), C = ratio of peak runoff rate to average rainfall rate over the time of concentration, i = rainfall intensity (inches/hr), and A = area of watershed under consideration (acres).

The principal idea of the rational method is that the peak rate of surface runoff, Q , from a subbasin is proportional to the subbasin area, A , and the average rainfall intensity, i , over a period of time just sufficient for all parts of the subbasin to contribute to the outflow (time of concentration). The runoff coefficient, C , is supposed to reflect all those characteristics of the watershed such as imperviousness and antecedent moisture condition, which affect the runoff rate.

Runoff coefficients for rural areas may be found in Table 3.2.1 and urban areas in Table 3.2.2. The time of concentration can be found using either Table 3.2.4 or an appropriate estimate. For a design return period, the rainfall intensity can be found using the method discussed in Sections 2.2.4 and 2.2.5, with the storm duration equal to the time of concentration.

When applied to a watershed with many subbasins, the formula is used to size each sewer pipe separately, considering all the upstream areas contributing to that pipe. Figure 7.2.1 has been presented to aid in the computations; Table 7.2.2 explains the headings for each column.

Examples 7.6.1 and 7.6.2 illustrate the use of the rational method for storm sewer design using Figure 7.2.1.

7.2.2 - Computer Programs for Storm Sewer Design

With the rapid development of personal computers, computations that once took many hours to complete are reduced to a few seconds of computer time. Consequently, it is now feasible to investigate the performance of alternate designs and the effect of varying design parameters.

One very popular and useful computer program for storm sewer design is the Illinois Urban Drainage Area Simulator (ILLUDAS) (Terstriep and Stall, 1974). This program allows for the design of up to 999 pipes or channels given the lengths and slopes of each reach. It requires no more data than that needed for the rational method but it enables the designer to investigate the effects of variations of storm frequency, duration, antecedent moisture condition and the temporal distribution of the rainfall with relative ease. In addition, the sizing of storage facilities is easily accomplished. This program which incorporates the basic principles of hydraulics and hydrology may be obtained along with the Users Manual from the Illinois State Water Survey, Urbana, Illinois (Bulletin #58).

A couple of computer programs used to design storm sewers are contained in the HYDRAIN system (FHWA, 1992). The Hydrain package contains seven programs; HYDRAIN, HYDRO - Hydrology, HYDRA - Storm drains, HYCLV - Culverts, WSPRO - Step backwater and bridge hydraulics, HY-8 - Culverts, and HYCHL - Roadside channels. The programs have editors that assist in developing input files and give explanations of the desired data.

Table 7.2.2

Storm Sewer Design Sheet - Rational Method, Explanation of Column Headings in Figure 7.2.1

Column 1	Identification number of the particular reach.
Column 2 & 3	Identification numbers of the upstream and downstream structures.
Column 4	Length of the reach under consideration.
Column 5	Runoff coefficient of the subbasin. This may be a weighted composite based on the type of cover in the subbasin.
Column 6	Area of the subbasin, in acres, entering the upstream structure.
Column 7	Product of the runoff coefficient and the subbasin area.
Column 8	Sum of the runoff coefficients and area products contributing to the pipe under consideration.
Column 9	Inlet time for the subbasin under consideration (time of concentration).
Column 10	Maximum inlet time and total travel time for the water from the most distant subbasin to contribute (the longest travel time when all are considered).
Column 11	Rainfall intensity for the storm design frequency and storm duration equal to the time of concentration (Column 10).
Column 12	Peak flowrate for the reach under consideration, $Q = i\sum CA$.
Column 13	Selected pipe diameter.
Column 14	Pipe slope between structures. This may be the slope of the ground or may be the slope necessary to achieve minimum velocity.
Column 15	Full flow capacity of the pipe with diameter selected in Column 13 and slope in Column 14, determined using Manning's Equation.
Column 16	Full pipe velocity found by $V = Q/A$.
Column 17	Travel time in the reach found by $t_r = L/60V$.
Column 18 & 19	Rim (ground) elevations of the upstream and downstream structures, respectively.
Column 20 & 21	Invert elevations of the pipe at the upstream and downstream structures found by subtracting the fall of the sewer from the upstream invert.
Column 22 & 23	Pipe cover for the upstream and downstream pipes. Found by subtracting the top of the pipe elevation from the rim elevation.

7.3 - HYDRAULICS OF CULVERTS

Culverts are special types of pipes employed in the conveyance of stormwater runoff because of the many types of flow which may occur. The flow is dependent upon the inlet geometry, slope, size, roughness and approach or tailwater conditions. In order to properly determine the quantity of flow which will be conveyed, all of the above need to be considered.

The various types of culvert flow which may be encountered are shown in Figure 7.3.1. If the outlet is submerged or if the outlet is not submerged but the headwater is high and the barrel is long, the culvert will flow full as shown in Types 1 and 2. A culvert which has the characteristics of Type 2 is termed "hydraulically-long". On the other hand, a culvert which is not long enough for flow to develop is called "hydraulically-short" and is categorized as Type 3. The culvert size, slope, entrance geometry and headwater and tailwater conditions determine whether a culvert is "hydraulically-long" or "-short". Charts are available to roughly distinguish between the two types. (Chow, 1988)

If the outlet and inlet are not submerged, and the tailwater is deeper than the critical depth (see Chapter 4, Section 4.2.2) the flow in the culvert is subcritical as shown in Figure 7.3.1 as Type 4. Two types of possible flow conditions exist when the tailwater depth is less than critical depth. The first type occurs when the depth of flow in the culvert is greater than the critical depth. For this condition the culvert is said to have "outlet-control", while a pipe depth less than critical depth will have supercritical flow and is under "inlet-control". These two conditions are shown as Types 5 and 6 in Figure 7.2. Table 7.3.1 summarizes each type in outline form.

Table 7.3.1
Summary of Culvert Flow (after Chow, 1988)

Condition	Flow Computations	Type (Figure 7.3 1)
A Outlet Submerged	full pipe flow, dependent on inlet geometry, etc	1
B Outlet Not Submerged		
1 Headwater greater than critical value		
(a) Culvert hydraulically long	full pipe flow	2
(b) Culvert hydraulically short	orifice pipe flow	3
2 Headwater less than critical value		
(a) Tailwater higher than critical depth	weir flow	4
(b) Tailwater lower than critical depth		
(i) Subcritical Slope	weir flow	5
(ii) Supercritical Slope	weir flow	6

The sizing procedure which is commonly employed is a method outlined in "Hydraulic Charts for the Selection of Highway Culverts" (Hydraulic Engineering Circular No. 5, 1965) and "Capacity Charts for the Design of Highway Culverts" (Hydraulic Engineering Circular No. 10, 1965), both by the Federal Highway Administration. The appropriate chart is dictated by the parameters of roughness, slope, headwater depth, tailwater depth, length and either inlet or outlet control. Rather than presenting the numerous charts in this manual, the reader is referred to the above circulars for the computation of culvert capacities. The circulars are presented in the computer program HY-8 (see Chapter 4, Section 4.5) which facilitates the lengthy computations required for the sizing procedure.

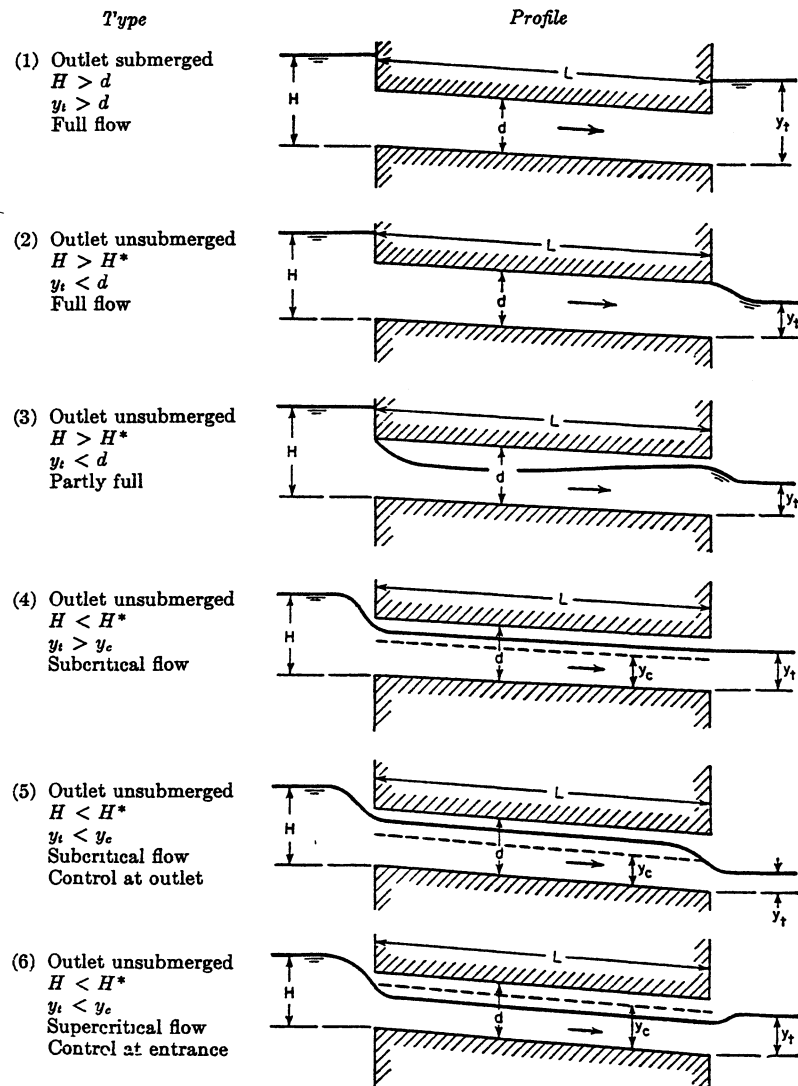


Figure 7.3.1
Types of Culvert Flow (Chow, 1988)

7.4 - CRITERIA FOR STORM SEWER DESIGN

This section outlines some criteria for the design of storm sewers.

1. Return Period: The minimum return period which should be used is 5 - 10 years for urban storm sewer systems. When there is a greater possibility of property damage, larger frequencies should be used. For roadways, the Indiana Department of Highways recommends the following return periods for culvert cross-drains and any type of facility in an underpass or depressed section of highway.

<u>Road System</u>	<u>Return Period</u>
Interstate	50 years
Primary	25 years
Secondary	10 years

2. Minimum Pipe Size: In order to minimize the potential for pipes to become clogged with debris, the minimum pipe size should be 12 inches. However, the diameter of a single lead from a catch basin or inlet conveying small runoff rates may be 10 inches.
3. Minimum Velocity: To minimize the potential for solids to be deposited in the invert of the sewer, the minimum pipe velocity should be 3.0 feet/second. Table 7.4.1 presents the minimum required slopes to achieve this velocity based on Manning's Equation for roughness coefficients of 0.013, 0.014 and 0.015.
4. Maximum Spacing of Manholes: Manholes should be placed whenever there is a junction or a change in grade or direction. A catch basin or inlet may be used instead of a manhole if it has sufficient size. The maximum spacing is usually between 300-400 ft. for smaller diameter pipe, but the distance may be as much as 500 ft. when the pipe diameter is large enough for a person to enter.
5. Pipe Elevation Changes in Structures: Whenever a pipe or pipes which enter a structure (manhole, catchbasin or inlet) have a different diameter than the outlet pipe, there must be a grade change between inverts. Commonly, the top elevation of the pipes are matched. Alternative rules dictate the matching of the hydraulic grade line or the 0.8D points of all pipes.
6. Minimum Depth of Storm Sewers: The top of the storm sewer should have a depth of cover which will protect it from live or dead loads which may be placed on it. Normally, the minimum depth should be 3 feet. Sometimes, however, a grade conflict between other sewers or water distribution systems may occur requiring the depth to be less than 3 feet. In this case extra strength pipe or special trench backfill procedures must be employed. The reader is referred to "Design Manual - Concrete Pipe" (Concrete Pipe Association of Indiana, 1974) or Modern Sewer Design,

(American Iron and Steel Institute, 1980) for information regarding pipe loadings.

7. Minor and Major Systems: The design of a storm sewer system should include an investigation of the impact of a high-frequency storm (i.e. 100 years) on the minor and major system. The minor system is comprised of the storm sewers, inlets, catchbasins etc. The major system includes the portion of the watershed which will become operative once the capacity of the minor system is exceeded. This includes the streets or other depressed areas which will act as open channels and convey the excess stormwater runoff to a natural watercourse or storage facility. If the resulting depth in these areas poses a danger to life or property, the capacity of the minor system should be increased.

Table 7.4.1
Minimum Pipe Slopes Necessary to Ensure a 3 ft/sec Velocity

Pipe Diameter	Percent Slope (ft/ft x 100)		
	$n = 0.013$	$n = 0.014$	$n = 0.015$
10	0.555	0.643	0.739
12	0.435	0.505	0.579
15	0.323	0.375	0.430
18	0.253	0.294	0.337
21	0.206	0.239	0.275
24	0.173	0.200	0.230
27	0.148	0.171	0.196
30	0.128	0.149	0.171
36	0.101	0.117	0.134
42	0.082	0.095	0.109
48	0.069	0.079	0.091
54	0.059	0.068	0.078
60	0.051	0.059	0.068
66	0.045	0.052	0.060
72	0.040	0.046	0.053
78	0.036	0.042	0.048

7.5 - MATERIALS USED FOR STORM SEWERS

This section briefly reviews some commonly used pipe materials for storm sewer systems. Table 7.5 lists the pipe materials commonly used for storm sewers and the sizes in which they are commercially available. The selection of one type over another is based on considerations of the durability of materials, installation cost and structural requirements.

Table 7.5.1
Commercially Available Materials and Sizes for Sewer Pipe

Nominal Diameter (inches)	CONC	RCCP	CSP	ESCP	ABS	PVC	CPVC	CI	VCC	CD	CAA
4	X		X	X		X	X		X	X	
6	X		X	X		X	X		X	X	X
8	X		X	X	X	X	X		X	X	X
10	X		X	X	X	X	X		X	X	X
12	X	X			X	X		X	X	X	X
14								X			
15	X	X			X	X			X	X	X
16								X		X	
18	X	X				X		X	X	X	X
20								X		X	
21	X	X				X			X	X	X
24	X	X				X		X	X	X	X
27		X				X			X		
30		X		X		X		X	X	X	X
33		X		X		X			X		
36		X		X		X		X	X		X
39											
42		X						X			X
48		X						X			X
54		X									X
60		X									X
66		X									
72		X									
78		X									
84		X									
90		X									
96		X									
102		X									
108		X									

Table 7.5.1 (cont'd)

<u>Symbol</u>	<u>Description</u>
X	Available in pipe size
CONC	Concrete pipe
RCCP	Reinforced concrete culvert and storm drain & sewer pipe
CSP	Concrete sewer and storm drain & culvert pipe
ABS	Acrylonitrile - Butadiene - Styrene composite sewer piping
PVC	Polyvinyl chloride (PVC) pipe
CPVC	Corrugated poly (vinyl Chloride) (PVC) pipe with smooth interior
CI	Cast iron pipe
VCC	Vitrified clay culvert
CD	Clay draintile (extra quality)
CAA	Corrugated aluminum alloy

NOTE: Verify pipe loading condition and consult with the pipe manufacture regarding pipe class and wall thickness.

7.6 - EXAMPLE PROBLEMS

Example 7.6.1

Determine the required pipe sizes for the hypothetical watershed shown in Figure 7.6.1 and the peak flowrate out of the watershed using the rational method and the design criteria presented in Section 7.5. The time of concentration, runoff coefficient and area are given for each subbasin. Also, the ground elevations at each structure and the distances between structures are shown. Assume the watershed is located near Lafayette, Indiana. Use Chen's Method (Section 2.2.5) to determine the rainfall intensities and match the tops of all pipes at structures.

Using Chen's Method and Example 6.2.1, the formula to determine the rainfall intensity is:

$$i_t^{T_r} = \frac{a'}{(60t + d')^{a'}} = \frac{55.77}{(60t + 9.2)^{0.78}}$$

The calculations are shown in Figure 7.6.2. The peak flowrate from the entire watershed is found to be 23.5 cfs. An explanation of each column and entry value for line number 3 is listed in Table 7.6.1.

It should be pointed out that the depths could have been lessened if the pipe had a larger diameter necessitating a smaller slope. There is always a trade-off between increased pipe costs with the larger diameter and the decrease in excavation costs or vice versa. This is where some economic analysis comes into play. A summary of methods to evaluate these costs was reported by Han, Rao and Houk (1980).

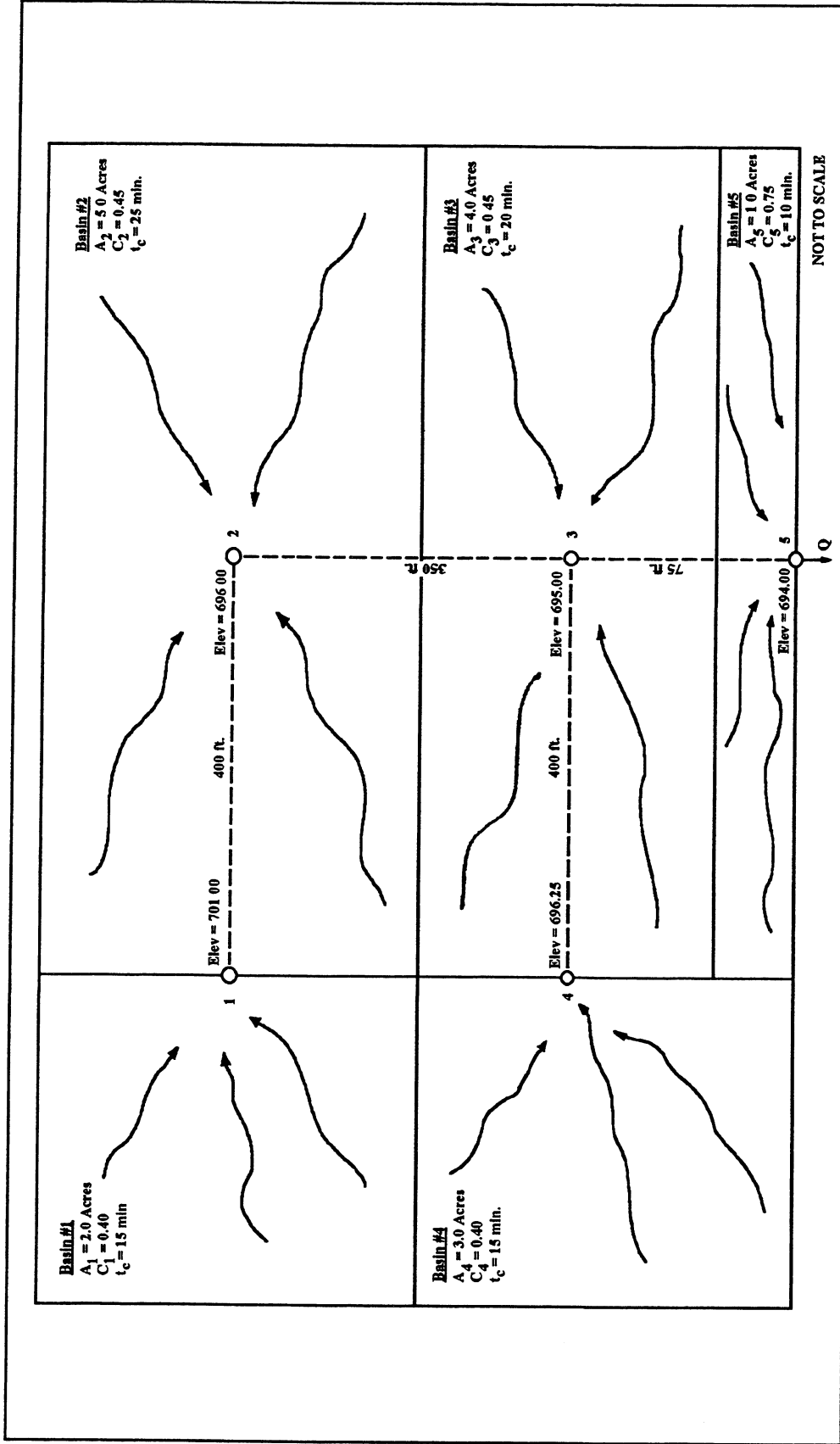


Figure 7.6.1
 Sketch of Hypothetical Watershed for Example 7.6.1

1	Line Number	
2	Upstream Manhole	(From Figure 7.6.1)
3	Downstream Manhole	(From Figure 7.6.1)
4	Length between structures	(From Figure 7.6.1)
5	C_i Runoff Coefficient for subbasin #3	(From Figure 7.6.1)
6	A_i Area (acres) of subbasin #3	(From Figure 7.6.1)
7	$C_i A_i$ Product of area and Runoff Coefficient for subbasin #3	(From Figure 7.6.1)
8	$\Sigma C_i A_i$ Sum of all Area-Runoff Coefficient products contributing to line #3	$(1.80 + 3.05 + 1.20 = 6.05)$
	Note. the 1.20 is from line #4.	
9	t_i Inlet time (minutes) for subbasin #3	(From Figure 7.6.1)
10	t_{cum} The longest Travel Time (minutes) to upstream structure #3	This is the larger of the two times:
	(a) The subbasin Inlet Time (20 minutes)	
	(b) The time of all upstream areas to contribute	$(25 + 1.25 \text{ minutes})$
11	1 Rainfall intensity (inches/hour) for Lafayette, Indiana for a ten-year return period and a storm duration equal to t_{cum} .	
	Q Peak Flow (cfs) into line #3, found by $Q = i \Sigma C_i A_i$	
12	Pipe Diameter (inches) selected to convey Q	
13	Pipe Slope (pct) at which the pipe selected above is placed	
14	Full Pipe Capacity (cfs) of the pipe with the selected diameter and slope determined by Manning's	
15	Equation (This must be greater than or equal to the peak flowrate, Q)	
	Velocity of flow (ft/sec) in pipe ($Q/A = V$)	
16	Travel Time (minutes) for the water to flow from Structure 3 to Structure 5.	
17	Rim Elevation Upstream	(From Figure 7.6.1)
18	Rim Elevation Downstream	(From Figure 7.6.1)
19	Invert Elevation Upstream	(From Line #2) - matching crowns
20	Invert Elevation Downstream	$689.5 - (75 \times 0.0075)$
21	Pipe Cover Upstream	$695.0 - 689.5 - 1.75$
22	Pipe Cover Downstream	$694.0 - 688.94 - 1.75$
23		

Example 7.6.2

This problem involves the design of the storm sewer system for Fair Oaks Estates, using the rational method. This subdivision, investigated by Burke and Gray (1979) is described below.

Fair Oaks Estates is a subdivision which was constructed in Carol Stream, Illinois, a suburb west of Chicago. (Some of the inputs have been varied from the as-built conditions.) The soil is classified as Type B and the topography is basically flat. The total area of the subdivision, which includes 30 lots and is 28% impervious, is 13.4 acres. The subdivision layout and the planned drainage system is shown on Figure 7.6.3. Inlets are located in the low spots and at the corners of intersections. The drainage subbasins which contribute to the inlets are also delineated in Figure 7.6.3. The storm sewer discharges into a detention pond located in the northeast corner.

The design requirements are:

1. The storm sewer must pass a ten-year storm, based on the intensity-duration-frequency curve for Chicago, Illinois.
2. The top of the pipe must have 3.5 ft. cover.
3. The full-pipe velocity must be greater than 2 ft/sec and less than 12 ft/sec.
4. The minimum pipe diameter is 12 inches.
5. The pipe is commercial reinforced concrete pipe with a Mannings n of 0.013.
6. The outfall pipe invert must be at or above 756.00 ft MSL.
7. When the pipe size changes at a structure, the top of the pipes shall be matched.
8. Subbasin inlet times shall be assumed.

From the plan drawings and topographic map, the ground elevations and pipe lengths were determined. The rational method was used to determine the required pipe diameters and slopes. The calculations are shown in Figure 7.6.4. An explanation of each column and entry value for line number 3 is listed in Table 7.6.2.

The schematic of the drainage system designed with the rational method is shown in Figure 7.6.5. The peak flow is found to be 16.90 cfs and the time of concentration for the basin is 27.78 min. The minimum pipe is 12 inches and the maximum diameter is 21 inches. The minimum pipe cover is 3.5 feet and the maximum is 4.74 feet. Velocities fall in the range of 2.04 ft/sec to 6.68 ft/sec.

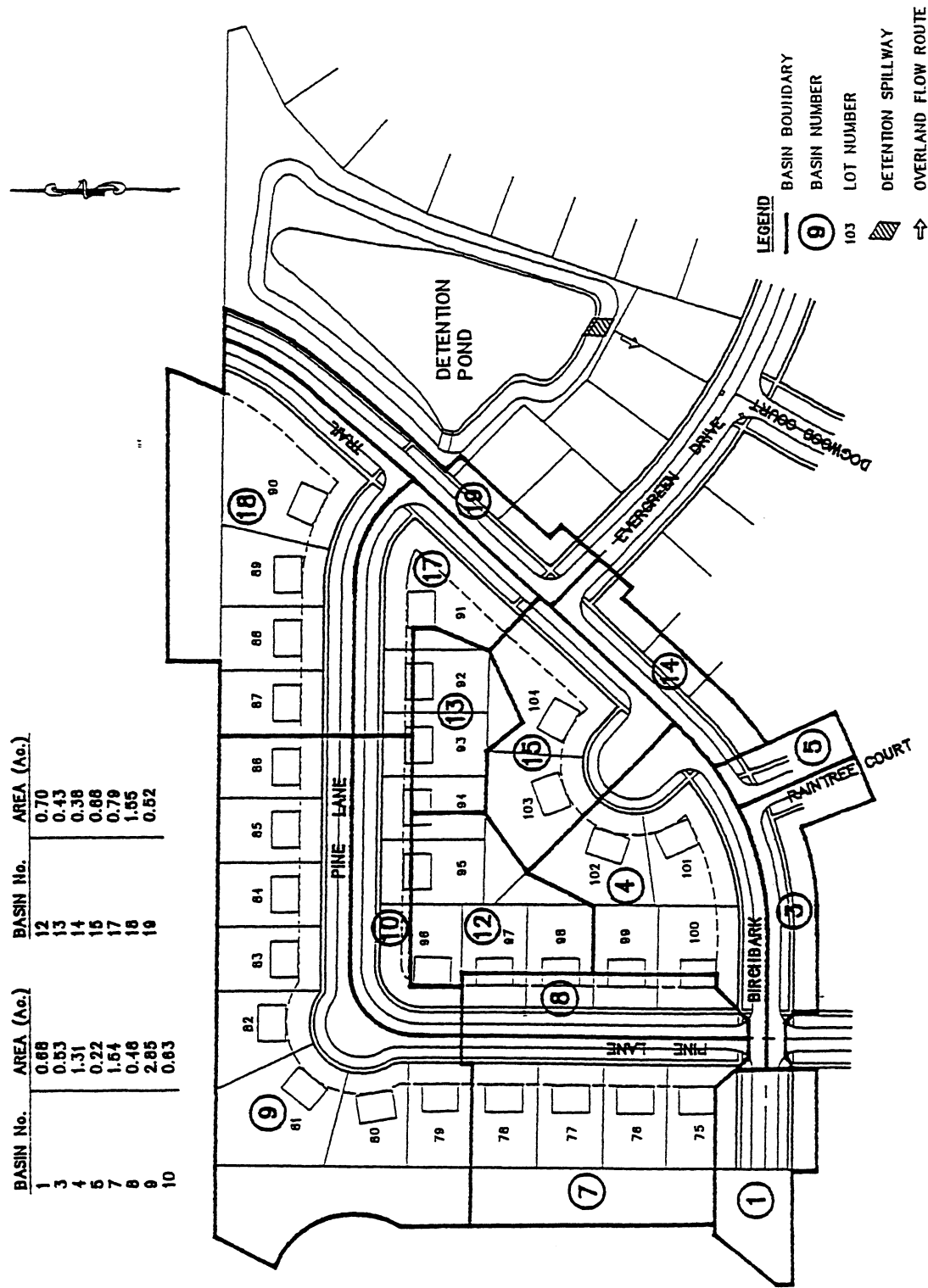


Figure 7.6.3
Fair Oaks Estates Hydrologic System

Table 7.6.2

Storm Sewer Design Sheet - Rational Method, Example 7.6.2 - Explanation of Entries

Column Number	Entry Value	Explanation - Comments
1	3	Line Number
2	19	Upstream Manhole
3	18	Downstream Manhole
4	76	Length between structures (From Figure 7 6 4)
5	0 44	C_j Runoff Coefficient for subbasin This is a weighted composite based on the type of cover in the subbasin
6	0 53	A_j Area (acres) of subbasin
7	0 23	$C_j A_j$ Product of area and Runoff Coefficient for subbasin (0 44 x 0 53 = 0 23)
8	0 41	$\Sigma C_j A_j$ Sum of all Area-Runoff Coefficient products contributing to line #3 (0 18 + 0 23 = 0 41)
9	20	t_j Time of concentration (minutes) for subbasin (Assumed to be 20 min)
10	20 9	t_{cum} The longest Travel Time (minutes) from the most distant basin to contribute (20 + 0 18 + 0 76 = 20 94 min)
11	3 94	t Rainfall intensity (inches/hour) for Chicago, Illinois for a ten-year return period and a storm duration of 20 94 min (3 94 inches/hour)
12	1 6	Q Peak Flow (cfs) into line #3, found by $Q = i \Sigma C_j A_j$, $Q = 3 94 (0 41) = 1 61$ cfs
13	12	Pipe Diameter (inches) selected to convey Q
14	0 010	Pipe Slope (pct) at which the pipe selected above is placed
15	3 62	Full Pipe Capacity (cfs) of the pipe with the selected diameter and slope determined by Manning's Equation (This must be greater than or equal to the peak flowrate, Q)
16	4 62	Velocity of flow (ft/sec) in pipe ($Q/A = V$)
17	0 27	Travel Time (minutes) for the water to flow from Structure 19 to Structure 18 ($t = L/60V = 0 27$ min)
18	766 14	Rim Elevation Upstream
19	765 88	Rim Elevation Downstream
20	761 61	Invert Elevation Upstream
21	760 83	Invert Elevation Downstream
22	3 53	Pipe Cover Upstream (Rim elev. - Top of Pipe elev)
23	4 05	Pipe Cover Upstream (Rim elev. - Top of Pipe elev)

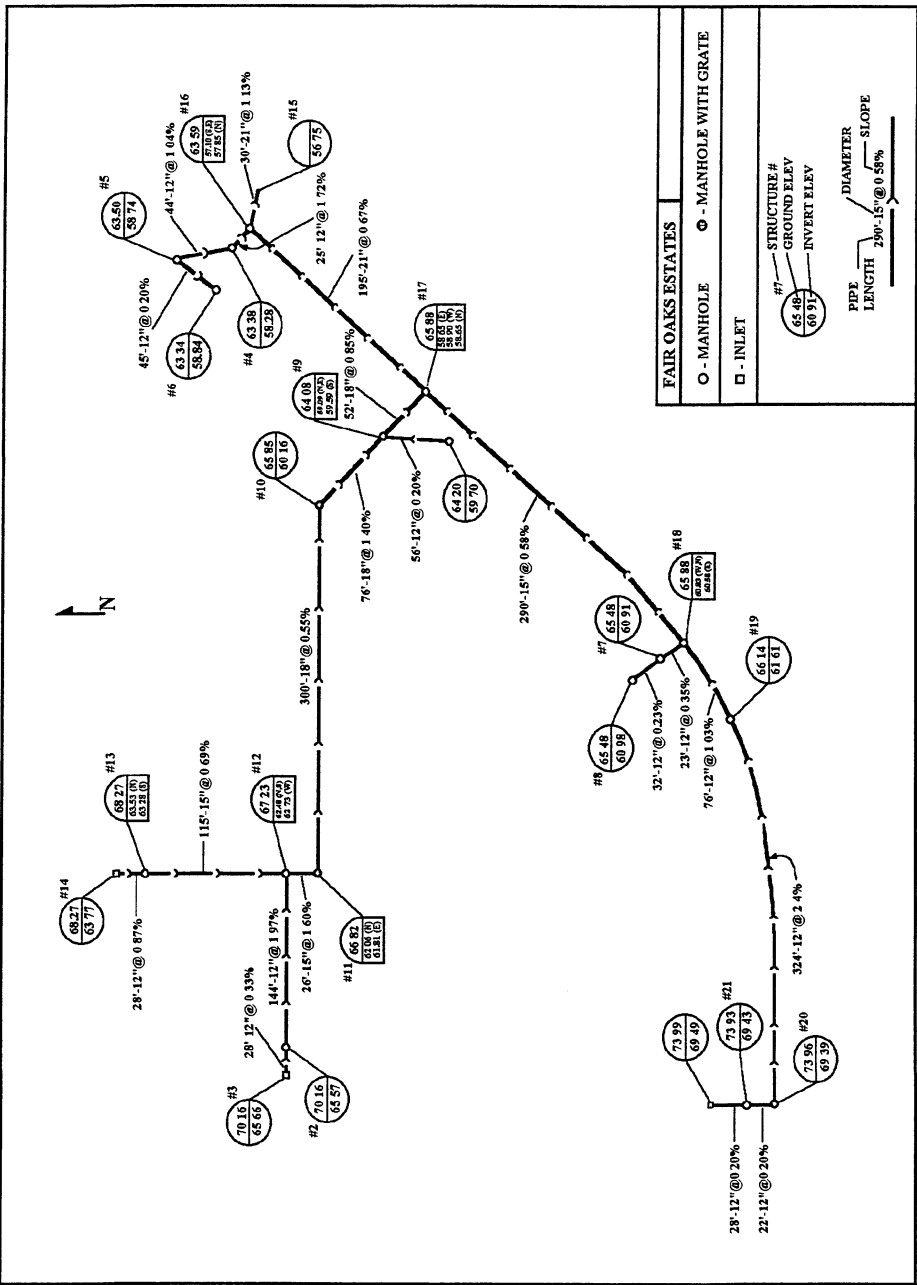


Figure 7.6.5
Layout of Storm Sewer Network System Designed by the Rational Method, Example 7.6.2

Table 7.6.3
Results of the Rational Method of Sewer System Design for Example 7.6.2 Using a 10-Year Return Period

Sewer Reach	Length (ft)	Drainage Area (acres)	C	Q_r^* (cfs)	D (inches)	S (%)	V (ft/sec)	Q_p^* (cfs)	Q_p / Q_r
21-20	221	0.63	0.28	0.71	12	0.2	2.03	1.60	2.26
20-19	324	-	-	0.71	12	2.40	7.06	5.55	7.82
19-18	76	0.53	0.44	1.61	12	1.03	4.62	3.62	2.25
8-7	32	1.31	0.22	1.68	12	0.23	2.18	1.71	1.00
7-18	23	0.22	0.47	2.07	12	0.35	2.69	2.11	1.02
18-17	290	-	-	3.65	12	0.58	4.02	4.93	1.35
14-13	28	-	-	3.33	12	0.87	4.24	3.33	1.00
13-12	115	0.63	0.49	4.46	15	0.69	4.38	5.38	1.21
3-2	28	1.54	0.33	2.04	12	0.33	2.61	2.05	1.00
2-12	144	0.48	0.54	3.06	12	1.97	6.38	5.01	1.64
12-11	26	-	-	7.22	15	1.60	6.68	8.19	1.13
11-10	300	0.70	0.25	7.82	18	0.55	4.42	7.82	1.00
10-9	76	0.43	0.25	8.04	18	1.40	4.55	12.46	1.55
1-9	56	0.39	0.39	0.61	12	0.20	2.03	1.60	2.63
9-17	52	0.88	0.38	9.69	18	0.85	5.49	0.72	1.00
17-16	195	-	-	12.91	21	0.67	5.41	13.00	1.00
6-5	45	0.79	0.45	1.40	12	0.20	2.03	1.60	1.14
5-4	44	1.55	0.37	3.65	12	1.04	4.64	3.65	1.00
4-16	25	0.52	0.50	4.67	12	1.71	5.95	4.67	1.00
16-15	30	-	-	16.90	21	1.13	7.02	16.90	1.00

* Q_r = Runoff from rational method
 Q_p = Capacity of the pipe flowing full

Chapter 7 - REFERENCES

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Chapter 8

COMPUTER APPLICATIONS

Chapter 8 - COMPUTER APPLICATIONS

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Chapter 8 - COMPUTER APPLICATIONS

For the design of hydraulic structures many lengthy computations are required. The task of performing hydrologic computations is simplified using computer models. Two computer models used to perform these tasks are TR-20 and HEC-1. TR-20 is a computerized version of the SCS method for calculating runoff volume resulting from rainfall, discussed in Section 6.2.2. A variety of unit hydrograph procedures are available to distribute the runoff in time. As mentioned in Chapter 6, computer models can more accurately and easily determine the distributed runoff. HEC-1 accomplishes the same basic tasks as TR-20 and has the additional features which are explained in Section 8.2.

This chapter utilizes the techniques learned in Chapters 2, 3 and 6 - precipitation, determination of peak runoff and volume, and stormwater storage. The chapter includes three examples for both models which illustrate the application of these programs. The same examples are used for both TR-20 and HEC-1.

8.1 - TR-20 PROGRAM

8.1.1 - Explanation of TR-20 and its Uses

The TR-20 computer program was prepared by the Soil Conservation Service (SCS) in 1977 and updated in 1982 (SCS, 1982) to assist in the hydrologic evaluation of storm events for water resource projects. The program is a single event model which computes direct runoff resulting from any synthetic or natural rainfall event of a specified duration. There is no provision for recovery of initial abstraction or infiltration during periods without rainfall. The program develops runoff hydrographs from excess precipitation and routes the flow through stream channels and reservoirs. It combines the routed hydrograph with those from tributaries and computes the peak discharges, their times of occurrence and the water elevations at any desired cross section or structure. The program provides the analysis of up to nine different rainstorm distributions over a watershed under various combinations of land treatment, flood control structures, diversions, and channel modifications. Such analysis can be performed on as many as 200 reaches and 99 structures in any one continuous run (SCS, 1982).

The program can be obtained from

National Technical Information Service
U S Department of Commerce
5285 Port Royal Road
Springfield, Virginia 22161
(703) 487-4600

8.1.2 - Summary of TR-20 Input Structure

The input data requirements are surprisingly few, with the amount of data depending on the complexity of the problem to be solved. If actual rainfall events are not going to be used, the depth of precipitation is the only meteorological input. For each subarea, the drainage area, runoff curve number, and the time of concentration are required; the antecedent soil moisture condition (i.e. AMC I, II, or III) can be specified, although the SCS now recommends only AMC II, the so-called average runoff condition (SCS, 1986). For each channel reach, the length is required along with the channel cross-section description, which is the discharge and end area data at each elevation; a routing coefficient, which is optional, may also be computed. If the channel routing coefficient is not given as input, it will be computed using a Modified Att-Kin (Attenuation-Kinematic) procedure.

The modified Att-Kin procedure for channel flood routing is based on the Att-Kin procedure described in TR-66 (SCS, 1982). It uses storage and kinematic models to attenuate and translate natural flood waves. For each channel reach, the Att-Kin procedure routes an inflow hydrograph through the reach using a storage model. This procedure then positions the peak in time and distorts the storage routed hydrograph by using a kinematic model. The storage and kinematic models are routed as a linear combination so that the outflow hydrograph satisfies the conservation of mass equation at the time to peak of the outflow hydrograph. The storage routing provides attenuation but does not describe translation; the kinematic routing provides translation and distortion but does not attenuate the peak (SCS, 1982). Channel routing has little, if any, effect on the attenuation of the peak discharge, unless the channel is roughly 200 feet wide and has a flowrate around 10,000 cfs. This is demonstrated in the last example

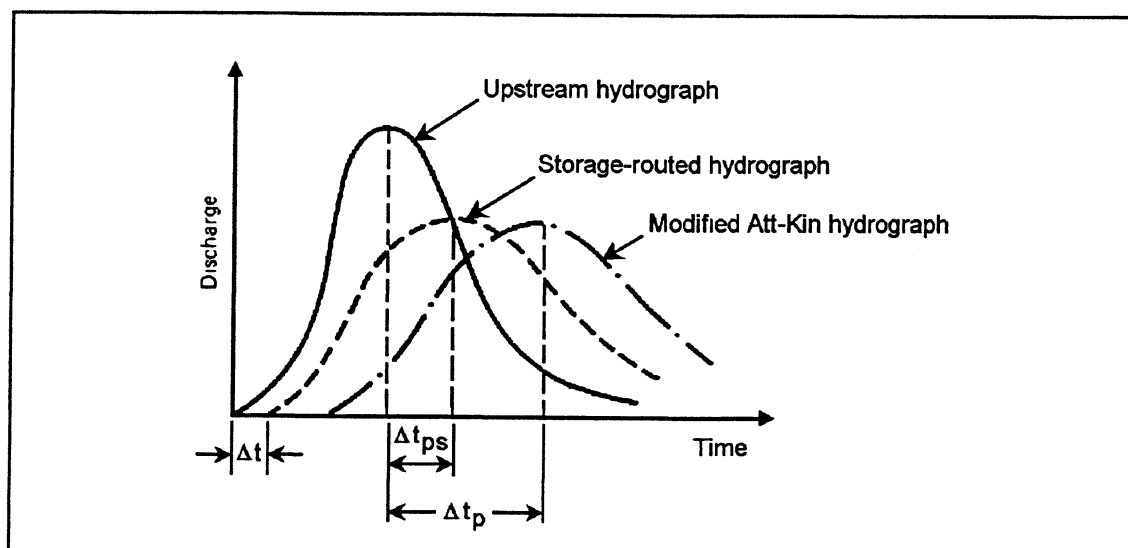


Figure 8 1 1

Hydrographs Resulting from Storage Routing & Kinematic Translation (SCS, 1982)

For each structure it is necessary to describe the outflow characteristics with the elevation-discharge-storage relationship. The time increment for all computations and any baseflow in a channel reach must be identified.

1. Input Data Structure

The input data for a TR-20 run consists of the following four general statement types: job control, tabular data, standard control statements, and executive control statements.

a. Job Control

The job control statements are the JOB statement, two TITLE statements, ENDDATA statement, the ENDCMP statement and ENDJOB statement. The JOB and TITLE statements must appear first and second, respectively, in a run and the ENDJOB statement must appear last. The ENDDATA statement separates the standard control and the tabular data from the executive control statements. The ENDCMP statement is used to signify the end of a pass through a watershed or a sub-watershed thereby allowing conditions to be changed by the user before further processing.

A blank form for listing each type of input data is included in Appendix D. Each line of data on the form is a record to be entered into the computer file. The 80 columns across the top of the form represent the 80 positions on a record. Each statement is used to perform a specific operation. The order of records determines the sequence in which the operations are to be performed. There can be up to 600 Standard Control statements for each TR-20 job.

A name appears in columns 4 through 9 of all lines except those containing tabular data. These names identify the type of operation performed or type of data entered. The digit in column 11 is the operation number and identifies the type of operation, which must be entered unless it is a blank.

b. Tabular Data

The tabular data serves as support data for the problem description. These data precede both the standard and executive control and have six possible tables:

1. *Routing coefficient table*: the relationship between the streamflow routing coefficient and velocity.
2. *Dimensionless hydrograph table*: the dimensionless curvilinear unit hydrograph ordinates as a function of dimensionless time. (see Chapter 3 for further explanation)
3. *Cumulative rainfall tables* (see Chapter 3)
4. *Stream cross-section data tables*: a tabular data summary of the water surface elevation-discharge-cross sectional end area relationship.

5. *Structure data table*: a tabular summary of the water surface elevation-discharge-reservoir storage relationship.
6. *Read-discharge hydrograph data table*: a hydrograph which is entered directly into the program as data.

For example, the data needed to support a *Cumulative rainfall table* includes a rainfall table number, time increment, runoff coefficient (if desired), and the cumulative rainfall values for the modeled rainfall event. The cumulative rainfall values used in the following example problems is the Huff Point Third Quartile distribution.

c. Standard Control Operations

Standard control consists of six subroutine operations.

- | | | |
|----|---------|--|
| 1. | RUNOFF | an instruction to develop a subbasin runoff hydrograph |
| 2. | RESVOR | an instruction to route a hydrograph through a structure or reservoir |
| 3. | REACH | an instruction to route a hydrograph through a channel reach |
| 4. | ADDHYD | an instruction to combine two hydrographs |
| 5. | SAVMOV: | an instruction to move a hydrograph from one computer memory storage location to another |
| 6. | DIVERT | an instruction for a hydrograph to be separated into two parts |

As an example, the data needed to support the standard RUNOFF control operation are the area, curve number, and time of concentration. This will be explained further for each example.

The operations are indicated on the input lines in columns 2, 4-9, and 11. The number 6 is placed in column 2, to indicate standard control. The name of the subroutine operation is placed in columns 4-9. The operation number, which is placed after the operation name (e.g., 1 for RUNOFF, 6 for DIVERT), is placed in column 11.

Column 61, 63, 65, 67, 69 and 71 of standard control records are used to specify the output. The individual output options are selected by placing a 1 in the appropriate column. If either the column is left blank or a zero is inserted, the corresponding option will not be selected. If only a summary table is desired, it is convenient to place SUMMARY in columns 51-57 in the JOB statement and leave all the output options on the standard control records blank. Table 8.1 summarizes the output options.

Table 8.1.1
Output Options for TR-20

<i>Output Option</i>	<i>A "1" in Column</i>	<i>Produces the Following Printout</i>
PEAK	61	Peak discharge and corresponding time of peak and elevation (maximum storage elevation for a structure).
HYD	63	Discharge hydrograph ordinates.
ELEV	65	Stage hydrograph ordinates (reach elevations for a cross section and water surface elevation for structures)
VOL	67	Volume of water under the hydrograph in inches-depth, acre-feet and ft ³ /sec-hours
SUM	71	Requests the results of the subroutine be inserted in the summary tables at the end of the job.

d Executive Control

The Executive Control has two functions: (1) to cause the Standard Control Statements to be executed and (2) to provide additional data necessary for processing. The Executive Control consists of six types of statements. They are LIST, BASFLO, INCREM, COMPUT, ENDCMP, and ENDJOB. The Executive Control statements are placed after the Standard Control statements and tabular data to which they pertain.

The Standard Control is used to describe the physical characteristics of the watershed. The Executive Control is used to prescribe the hydrologic conditions of the watershed including the baseflow. The performance for the Executive Control is directed by the COMPUT statement. Its purpose is to describe the rainfall and the part of the watershed over which that rainfall is to occur (SCS, 1982).

For the example problems discussed in this chapter, the INCREM, COMPUT, ENDCMP, and ENDJOB will be the only statements required to fulfill the operations requested.

2 Preparation of Input Data

Preparation of input data can be divided into the following requirements and functions.

- 1 Prepare a schematic drawing that conveniently identifies the locations, drainage areas, CNs, t_c s, and reach lengths for the watershed. It should display all alternate structural systems together with the routing and evaluation reaches through which they are to be analyzed.

2. Establish a Standard Control list for the watershed.
3. Compile the tabular data to support the requirements of the Standard Control list. This may consist of Structure Data, Stream Cross Section Data, Cumulative Rainfall Data, and the Dimensionless Hydrograph Table.
4. Establish the Executive Control statements that describe each storm and alternative situation that is to be analyzed through the Standard Control list.

a. Calculations

For a large watershed it may be necessary to divide the watershed into subbasins. Each subbasin is determined by finding the different outlet points or design points within the watershed, then finding the area contributing to those points.

- (1) **Area** - The area of each subbasin must be calculated in square miles (mi^2). This can be done with a digitizer, a planimeter, or simply by counting squares.
- (2) **Curve Number** - The Curve Number (CN) must be computed for each subbasin. As described in Chapter 3, the runoff curve number is a dimensionless number of 100 or less that relates runoff to the soil-cover complex of the watershed. Higher curve numbers mean greater runoff. The tables for calculating the curve number can be found in Table 3.8. A worksheet, from the TR-55 manual (SCS, 1986), for calculating the curve number is provided (Appendix D) which multiplies a subbasin's area(s) times the curve number(s) for the respective subbasin in order to compute a weighted value.
- (3) **Time of Concentration** - Following the curve number, the time of concentration (t_c) is calculated for each subbasin. The time of concentration is defined as the time required for a hypothetical particle of water to flow hydraulically from the most distant point in the watershed to the outlet point or design point. The t_c is calculated using SCS TR-55 methodology, which is explained in Appendix D. This appendix also contains blank worksheets used for computing the time of concentration. The methodology divides flows into three types: sheet flow, shallow concentrated flow, and channel flow. The travel time must be computed for each type of flow that is found. These times are then added together to find the total t_c .

b. Forms for Input Data

Blank forms for input data are in Appendix D. The first example problem demonstrates how the forms are completed. The process is straightforward when using the prepared forms because of the instructions given and the pre-selected columns for the data.

8 1.3 - TR-20 Example Problems

A watershed located near Lafayette, Indiana is modeled to assess the watershed peak runoff rates for existing conditions. The first example determines the runoff from one subbasin. The second example routes the discharge from the subbasin through a culvert. Lastly, the third example incorporates a reach and additional subbasin to determine the peak discharge from the watershed at the outlet. The examples are successive and the delineation of the watershed is shown on Exhibit 8 2 1.

Example 8.1.1

1. Description

Referring to Exhibit 8.2.1, Subbasin 1 (SA1) is modeled to determine the peak runoff for present conditions using the 12-hour, 100-year rainfall depth. This area comprises all of the watershed that is on the west side of Interstate 65 before it passes under the Interstate. By this example, it is intended to explain the basic input for any TR-20 run

2 Hydrological Input Data

The cumulative rainfall data is the Huff Point Third Quartile rainfall distribution. To find the 12-hour rainfall depth for the 100-year recurrence interval, Chen's method (Chapter 2) is consulted. The point rainfall depth is computed as 5.48 inches for the Lafayette area.

3 Calculations

Note: All references made to Tables and Figures in the following calculations refer to the SCS TR-55 tables and figures in Appendix D.

Area - The area of subbasin 1 is computed to be 461 acres or 0.72 mi², by the use of a digitizer.

Curve Number - In order to determine the curve number (CN) representing this subbasin, the existing soil group can be found from the SCS County Soils Map, or if it is presently developed use the corresponding CN. There are many different uses for this subbasin which are used to calculate a composite curve number. For example, 13.8 acres (3.0%) of the subbasin is commercial area. From Table 3.3.3 the curve number for commercial area on hydrologic soil group C is 94. Similarly, the other land uses are located on Tables 3.3.3 and 3.3.4 and their respective curve numbers are found. The total composite curve number and percentage of area is 8423, which corresponds to a composite CN of 84. Table 8 1 2 shows these calculations presented in a tabular format.

Table 8.1.2
SCS TR-55 Calculations for Example 8.1.1

Worksheet 2: Runoff curve number and runoff

Project ROSS DITCH - EXAMPLE 8.1.1 By TTB Date 8/94
 Location TIPPECANDE COUNTY Checked RSG Date 8-94
 Circle one: Present Developed SUBBASIN 1 (SA 1)

1. Runoff curve number (CN)

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN ^{1/}			Area <input type="checkbox"/> acres <input checked="" type="checkbox"/> mi ² <input type="checkbox"/> %	Product of CN x area
		Table 2-2	Fig. 2-3	Fig. 2-4		
C	ROW CROPS, GOOD	82			61.7	5059.4
	COMMERCIAL	94			30	282.
C	BRUSH, FAIR	70			3.3	231.
	OPEN SPACE, GOOD	74			2.6	192.4
	IMPERVIOUS, WATER	100			17.4	1740
	IMPERVIOUS, ROADS	98			1.4	137.2
B	ROW CROPS, GOOD	75			9.0	675.
	OPEN SPACE, GOOD	61			0.4	24.4
B	RESIDENTIAL, 1 ACRE	68			1.2	81.6
Totals =					100	8423

^{1/} Use only one CN source per line.

$$CN \text{ (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{8423}{100} = 84.23$$

Use CN = 84

2. Runoff

Frequency yr
 Rainfall, P (24-hour) in
 Runoff, Q in
 (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)

Storm #1	Storm #2	Storm #3

Time of Concentration - Finding the path of a hypothetical particle of water to flow hydraulically from the most distant point of Subbasin 1 to the point of discharge reveals 300 ft. of sheet flow and 2000 ft. of shallow concentrated flow over unpaved surface. Table 8.1.3 shows the calculations for the time of concentration using the SCS TR-55 worksheet. The total time of concentration for subbasin 1 is 1 50 hrs

4. Computer Input

The following section demonstrates where, for this example, the information is input to the computer. The input can be implemented with the use of a text editor to begin a new file or by editing an old file to input the desired information.

- (1) The first input is the JOB and TITLE. Describe the name of the project and inform the computer what operation, if any, is going to be performed and printed. For this example NOPLOTS is entered in columns 61-67, which indicates that cross section discharge-area plots are not desired. The two TITLE statements were used to describe the example and rainfall information, as shown on Figure 8.1.2.
- (2) The cumulative rainfall table is used to describe the rainfall distribution over the watershed. The statement describing this operation is RAINFL. There are six standard rainfall distributions as shown in Tables 1 through 6 in Appendix D which have been preloaded into TR-20. A list of these tables may be obtained by putting a '7' in column 2 and LIST in columns 4 through 7 on a line right before the COMPUT statement. The user does not need to input these tables, but needs only to use the proper rain table identification number on the COMPUT statement, in column 11. The user may override any of these tables by entering a new RAINFL table. For this example the rainfall distribution is the Huff Point Third Quartile. The distribution should be dimensionless so that the desired storm duration(s) may be input in the COMPUT statement(s). To make the distribution dimensionless, take the inverse of the total number of increments. Not including the 0 0 and extra 1.0's, the total number of points is 20. Accordingly, a '0.05' time increment is placed in columns 25 through 36. Rainfall depths in inches may also be used by changing the data code in column 2, of the RAINFL statement, to '8'.
- (3) The Standard Control statements are used to describe the runoff, reservoir routing, and combining of the hydrographs. For this example, the runoff from subbasin 1 is described by the RUNOFF statement with the proper codes, described previously. This is the only standard control statement needed for this operation. ENDATA indicates that the input information for the runoff is complete. Comments in columns 73 through 80 are helpful reminders.

Table 8.1.3

Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project ROSS DITCH - EXAMPLE 8.1.1 By TTB Date 5/94

Location TIPPECANOE COUNTY Checked RSB Date 8-94

Circle one: Present Developed

Circle one: T_c T_c through subarea

SUBBASIN 1 (SA 1)

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

Segment ID

1. Surface description (table 3-1)

2. Manning's roughness coeff., n (table 3-1) ..

3. Flow length, L (total L < 300 ft) ft

4. Two-yr 24-hr rainfall, P_2 in

5. Land slope, s ft/ft

6. $T_t = \frac{0.007 \text{ (nL)}^{0.8}}{P_2^{0.5} s^{0.4}}$ Compute T_t hr

Compute T_r hr

Shallow concentrated flow

Segment ID

7. Surface description (paved or unpaved)

8. Flow length, L ft

9. Watercourse slope, s ft/ft

10. Average velocity, V (figure 3-1) ft/s

11. $T_t = \frac{L}{3600 V}$ Compute T_t hr

Compute T_r hr

Channel flow

Segment ID

12. Cross sectional flow area, a ft^2

13. Wetted perimeter; p_w ft

14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ft

15. Channel slope, s ft/ft

16. Manning's roughness coeff., n

17. $V = \frac{1.49 r^{2/3} s^{1/2}}{n}$ Compute V ft/s

Compute V ft/s

18. Flow length, L ft

19. $T_t = \frac{L}{3600 \text{ v}}$ Compute T_t hr

Compute T_t hr

20. Watershed or subarea T_c or T_f (add T_f in steps 6, 11, and 19) hr

- (4) The Executive Control statement is used to indicate which computations are desired. This example desires to determine the runoff from Area 1 for the 10-year recurrence interval. The INCREM statement causes all the hydrographs generated within the TR-20 program to have the time increment specified in columns 25-36, unless the main time increment is changed by a subsequent INCREM statement. The criterion used for determining the main time increment is that it should be made short enough to describe the hydrographs for smaller watersheds and large enough that, when multiplied by the number of coordinates, extends past the peak periods of larger hydrographs so as not to significantly truncate the hydrograph volume used in reach routing (SCS, 1982)

The '8' in column 61 of the COMPUT statement refers to rainfall table 8, from the Cumulative Rainfall Table. The AMC (Antecedent Moisture Condition - the degree of wetness of the soil for the watershed at the beginning of a storm) number must be given in column 63 for each COMPUT (computation). For this example the AMC was 2, the recommended average runoff condition (SCS, 1986) The starting times, rainfall depth, and rainfall duration are given for each recurrence interval. The rainfall depth for the desired event, as explained on page 8-7, is placed in columns 37 through 48 A COMPUT and an ENDCMP statement is necessary for each recurrence interval. ENDJOB is then used to signify that all the information has been given and the calculations are to begin. The Executive Control data is shown on Figure 8.1.2

JOB TR-20										NOPLOTS									
TITLE		EXAMPLE 8.1.1 - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN																	
TITLE		EXISTING CONDITIONS - SUBBASIN 1																	
5	RAINF	8																	
				.05															
8			0.0	.03	.06	.09	.12												HUFF 3RD
8			.15	.19	.23	.27	.32												QUARTILE
8			.38	.45	.57	.70	.79												DISTRIB.
8			.85	.89	.92	.95	.97												
8			1.	1.	1.	1.	1.												
9	ENDTBL																		
6	RUNOFF	1	1	1	0.72	84.	1.50	1	0	0	1	0	1	SA	1				
	ENDATA																		
7	INCREM	6		0.25															
7	COMPUT	7	1	1	0.00	5.48	12.	8	2	1	2	100-YR							
	ENDCMP 1																		
	ENDJOB 2																		

Figure 8.1.2
TR-20 Input for Example 8.1.1

5 Output

The type and amount of output can be controlled by input options on the JOB record and by the output options on the Standard Control records. For this example, NOPLOTS was placed in columns 61-67 of the JOB record. The RUNOFF statement requested to have the peak discharge, volume of water under the hydrograph, and a summary table of the results, because a "1" was placed in columns 61, 67, and 71 in this statement. The computer generated output is included below.

6. Summary/Explanation

The summary table on the last page of the output clearly summarizes the information obtained. From this table it can be seen that, for a rainfall event with a 100-yr. recurrence interval, the amount of excess rainfall from subbasin 1 is 3.71 inches for a 12-hour duration. The peak discharge occurs after 844-hours and has a flowrate of 39113 cfs.

```
*****80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****

JOB TR-20                                NOPLOTS
TITLE  EXAMPLE 8.1.1 - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
TITLE  EXISTING CONDITIONS - SUBBASIN 1
5 RAINFL 8 .05
8      0.0 .03 .06 .09 .12 HUFF 3RD
8      .15 .19 .23 .27 .32 QUARTILE
8      .38 .45 .57 .70 .79 DISTRIB.
8      .85 .89 .92 .95 .97
8      1. 1. 1. 1. 1.
9 ENDTBL
6 RUNOFF 1 1 1 0.72 84. 1.50 1 0 0 1 0 1 SA 1
  ENDDATA
7 INCREM 6 0.25
7 COMPUT 7 1 -1 0.00 5.48 12. 8 2 1 2 100-YR
  ENDCMP 1
  ENDJOB 2
0*****END OF 80-80 LIST*****

EXECUTIVE CONTROL OPERATION INCREM                                RECORD ID
+
+ MAIN TIME INCREMENT = .25 HOURS

EXECUTIVE CONTROL OPERATION COMPUT                                RECORD ID 100-YR
+
+ FROM XSECTION 1
+ TO XSECTION 1
  STARTING TIME = .00 RAIN DEPTH = 5.48 RAIN DURATION= 12.00 RAIN TABLE NO.= 8 ANT. MOIST. COND= 2
  ALTERNATE NO.= 1 STORM NO.= 2 MAIN TIME INCREMENT = .25 HOURS
```

Figure 8.1.3(a)
TR-20 Output for Example 8.1.1

OPERATION RUNOFF CROSS SECTION 1

PEAK TIME(HRS) PEAK DISCHARGE(CFS) PEAK ELEVATION(FEET)
8.44 391.13 (RUNOFF)

RUNOFF VOLUME ABOVE BASEFLOW = 3.71 WATERSHED INCHES, 1723.35 CFS-HRS, 142.42 ACRE-FeET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP

RECORD ID

+ COMPUTATIONS COMPLETED FOR PASS 1

EXECUTIVE CONTROL OPERATION ENDJOB

RECORD ID

1

TR20 XEQ 08-12-94 12:26
REV PC 09/83(.2)

EXAMPLE 8.1.1 - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
EXISTING CONDITIONS - SUBBASIN 1

JOB 1 SUMMARY
PAGE 1

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION			RUNOFF AMOUNT (IN)	PEAK DISCHARGE			
						BEGIN (HR)	AMOUNT (IN)	DURATION (HR)		ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 1 STORM 2													
+ XSECTION 1	RUNOFF	.72	8	2	.25	.0	5.48	12.00	3.71	---	8.44	391.13	543.2
1													

TR20 XEQ 08-12-94 12:26
REV PC 09/83(.2)

EXAMPLE 8.1.1 - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
EXISTING CONDITIONS - SUBBASIN 1

JOB 1 SUMMARY
PAGE 2

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS.....
0 XSECTION 1	.72	2
+ ALTERNATE 1		391.13
LEND OF 1 JOBS IN THIS RUN		

Figure 8 1.3(b)
TR-20 Output for Example 8 1.1

Example 8.1.2

1. Description

The runoff from Subbasin 1 flows under Interstate 65. However, because the culverts are small relative to the flow, there is ponding of runoff. To represent this, the runoff from Subbasin 1 is routed through twin culverts under Interstate 65. A stage-storage-discharge relationship is required for the structure, and the attenuation effects are evaluated. Table 8.1.4 shows the stage--discharge relationship obtained from the HY-8 example in Chapter 4 (Example 4.5 1). The storage is determined from digitizing the existing contours of the subbasin and interpolating.

Table 8.1 4
Stage-Storage-Discharge Relationship for Culverts and Pond in Example 8 1.2

<i>Stage (feet)</i>	<i>Discharge (cfs)</i>	<i>Storage (acre-ft)</i>
654 16	0	0.0
654 75	5	98.1
655 08	10	117.2
656 10	15	195.6
656 16	20	201.5
656 29	30	214.2
656 43	40	228.0
656 59	50	243 7

2 Computer Input

- (1) The TITLE statement is adjusted to show the proper example number and description.
- (2) The rainfall table from Example 8.1 1 is used.
- (3) After the ENDTBL statement of the rainfall table, the structure table for the depressional area is added. The initial values for the stage-storage-discharge relationship must have the elevation of the invert of the outlet structure and a 0.0 value for the discharge which is in Data Field No. 2 All data entered in this table must have decimal points and must increase between successive lines of data. The STRUCT table

must have a number, placed in columns 16 and 17 of the first row of the STRUCT statement to correspond with the structure number on the RESVOR Standard Control statement. The number chosen for these culverts is '01'.

- (4) The RUNOFF Standard Control statement for subbasin 1 is the same as in the previous example
- (5) A RESVOR Standard Control statement routes the hydrograph from subbasin 1 through the culverts. This is done by placing 01 (the depressional area structure number) in columns 19 and 20. The routed hydrograph must be placed in a storage location, so any storage number between 1-7, not already used, is placed in column 23. Therefore, a 2 is placed in column 23.
- (6) The INCREM and COMPUT Executive Control statements from Example 8.1.1 are used. In order for the flow through the culverts to be analyzed, the computer must be informed that the last cross-section is the culvert structure, not subbasin 1. This is done by placing the cross-section number of the culvert in columns 20 and 21 of the COMPUT statement. With these statements, the peak discharge rate and time is computed, as well as the peak elevation in the culvert. Again, ENDJOB is the last statement. The input file is shown on Figure 8.1.4.

3 Output

The output format is the same as Example 8.1.1 because the output options in columns 61-70 are the same, with the addition of the options chosen for the RESVOR statement. A condensed version of the output is shown on Figure 8.1.5.

5 Summary/Explanation

Similar to Example 8.1.1, the summary table on the last page of the output summarizes most of the information. This table contains the peak runoff rate for subbasin 1 and the peak flow and elevation through the culverts. The culvert elevation is checked to determine if the size is sufficient to avoid overtopping. The peak discharge elevation is 656.84 feet for the 100-year storm. This indicates that the flow does not overtop I-65 for the storm event modeled.

Table 8.1.5
Summary of Example 8.1.2

<i>LOCATION</i>	<i>PEAK DISCHARGE (cfs)</i>	<i>RUNOFF AMOUNT (inches)</i>	<i>WATER SURFACE ELEVATION (feet)</i>	<i>TIME TO PEAK (hrs)</i>
Subbasin 1	391.13	3.71	-	8.44
Culverts	19.56	1.95	656.15	13.73

```

JOB TR-20
TITLE EXAMPLE 8.1.2 - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
TITLE EXISTING CONDITIONS - SUBBASIN 1 AND TWIN CULVERTS
5 RAINFL 8 .05
8 0.0 .03 .06 .09 .1 HUFF 3RD
8 .15 .19 .23 .27 .32 QUARTILE
8 .38 .45 .57 .70 .79 DISTRIB.
8 .85 .89 .92 .95 .97
8 1. 1. 1. 1. 1.
9 ENDTBL
3 STRUCT 01
8 654.0 12.7 58.5 DEPRESS-
8 654.17 0.0 67.5 IONAL
8 654.75 5.0 98.1 AREA
8 655.08 10.0 117.2
8 656.10 15.0 195.6
8 656.16 20.0 201.5
8 656.29 30.0 214.2
8 656.43 40.0 228.0
8 656.59 50.0 243.7
9 ENDTBL
6 RUNOFF 1 1 1 0.72 84. 1.50 1 0 0 1 0 1 SA 1
6 RESVOR 2 01 1 2 654.17 1 0 0 1 0 1 DEPRESS
ENDATA
7 INCREM 6 0.25
7 COMPUT 7 1 01 0.00 5.48 12. 8 2 1 2 100-YR
ENDCMP 1
ENDJOB 2

```

Figure 8.1 4
TR-20 Input for Example 8 1.2

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
 (A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
 A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION			RUNOFF AMOUNT (IN)	PEAK DISCHARGE				
						BEGIN (HR)	AMOUNT (IN)	DURATION (HR)		ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)	
ALTERNATE 1 STORM 2														
+														
XSECTION	1	RUNOFF	.72	8	2	.25	.0	5.48	12.00	3.71	---	8.44	391.13	543.2
STRUCTURE	1	RESVOR	.72	8	2	.25	.0	5.48	12.00	1.95	656.15	13.73	19.56	27.2
1														

TR20 XEQ 08-12-94 12:27
 REV PC 09/83(.2)

EXAMPLE 8.1.2 - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
 EXISTING CONDITIONS - SUBBASIN 1 AND TWIN CULVERTS

JOB 1 SUMMARY
 PAGE 2

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS..... 2
0 STRUCTURE 1	.72	
+		
ALTERNATE 1		19.56
0 XSECTION 1	.72	
+		
ALTERNATE 1		391.13
1END OF 1 JOBS IN THIS RUN		

Figure 8.1 5
 TR-20 Output for Example 8.1 2

Example 8.1.3

1. Description

The entire watershed is analyzed to determine the peak runoff from the 0.87 square miles. This example incorporates the area from Example 8.1.2 and routes the outflow from the culverts through a ditch and adds the runoff from subbasin 2 (SA 2), as shown on Exhibit 8.1.

The hydrologic input data from the previous examples is used. This refers to the rainfall data for the Huff Third Quartile rainfall distribution and the 5.48 rainfall depth corresponding to a 12-hour 100-year recurrence interval in Lafayette. In addition, the 12-hour 10-year and 24-hour 10 and 100-year rainfall events are computed to demonstrate the simplicity of adding additional events.

2. Calculations

The calculations for subbasin 2 are performed in the same manner as subbasin 1 was calculated. The area is digitized and the curve numbers are computed using the appropriate land use. The time-of-concentration is calculated using SCS TR-55 methodology. Tables 8.1.6 and 8.1.7 contain the information for the computation of the CN and t_c , respectively.

Table 8.1.6
Curve Number Calculation for Subbasin 2

<i>Drainage Area</i>	<i>Area (%)</i>	<i>Land Use (Hydrologic Group)</i>	<i>CN</i>	<i>Area * CN</i>
2	38.0	Row Crop, Good (B)	75	2850.0
	5.3	Open Space, Good (B)	61	323.3
	1.4	Woods, Good (B)	55	77.0
	1.7	Brush, Good (B)	48	81.6
	8.0	Impervious, Roads	98	784.0
	26.0	Row Crop, Good (C)	82	2132.0
	14.2	Open Space, Good (C)	74	1050.8
	3.4	Woods, Good (C)	70	238.0
	2.0	Brush, Good (C)	65	130.0
TOTAL	100			7666.7
Composite Curve Number				77

Table 8.1.7
SCS-TR55 t_c Calculations for Subbasin 2

Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project ROSS DITCH - EXAMPLE 8.1.3 By TTB Date 8/94

Location TIPPELANCE COUNTY Checked RSB Date 8-94

Circle one: Present Developed

Circle one: T_c T_t through subarea SUBBASIN 2 (SA2)

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

Segment ID

1. Surface description (table 3-1)

2. Manning's roughness coeff., n (table 3-1) ..

3. Flow length, L (total $L \leq 300$ ft) ft

4. Two-yr 24-hr rainfall, P_2 in

5. Land slope, s ft/ft

6. $T_c = \frac{0.007 (nL)^{0.8}}{P_2^{0.5} s^{0.4}}$ Compute T_t hr

I	
WOODS	
0.40	
200	
3.00	
0.01	
0.85	

= 0.85

Shallow concentrated flow

Segment ID

7. Surface description (paved or unpaved)

8. Flow length, L ft

9. Watercourse slope, s ft/ft

10. Average velocity, V (figure 3-1) ft/s

11. $T_t = \frac{L}{3600 V}$ Compute T_t hr

II	
UNPAVED	
2500	
0.008	
1.44	
0.48	

= 0.48

Channel flow

Segment ID

12. Cross sectional flow area, a ft²

13. Wetted perimeter, p_w ft

14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ft

15. Channel slope, s ft/ft

16. Manning's roughness coeff., n

17. $V = \frac{1.49 r^{2/3} s^{1/2}}{n}$ Compute V ft/s

18. Flow length, L ft

19. $T_t = \frac{L}{3600 V}$ Compute T_t hr

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) hr

= 1.33

3. Computer Input

The input file from Example 8.1.2 is edited to include the information for the reach and additional subbasin. The Standard Control statements, REACH, RUNOFF and ADDHYD, are required. The ADDHYD statement combines two hydrographs. For example, if there were three subbasins to be routed through the culverts, the hydrographs would be combined by two ADDHYD statements and then routed through the culvert as one hydrograph.

The routing is described by a REACH statement which refers to XSECT 005. The XSECT is the stage-storage-discharge relationship for the 1500 ft. channel. The relationship is computed given the channel cross-section shown in Figure 8.1.6. The RUNOFF statement is then used to describe the runoff from subbasin 2. Finally, the hydrographs from the REACH and subbasin 2 runoff are added together with the ADDHYD statement.

The Executive Control statements are the same as those used in the previous two examples, except the last cross-section number is placed in columns 20 and 21. The COMPUT statements require the input of the initial and final cross-section that are to be analyzed. Because the ADDHYD is the last Standard Control statement, its cross-section number (columns 14 and 15 of the ADDHYD statement) is placed in columns 20 and 21 of the COMPUT statement. Three more COMPUT and ENDCMP statements are added to compute the 3 additional events. Either the alternate or storm number is changed on each COMPUT statement.

4. Output

The type and amount of output can be controlled by input options on the JOB record and by the output options on the Standard Control records. The same type of output was called for as in the previous examples. The RUNOFF statement requested to have the peak discharge, volume of water under the hydrograph, and a summary table produced in the printout of the results, because a "1" was placed in columns 61, 67, and 71 in this statement. The computer generated output is included after this example on Figure 8.1.8.

5. Summary/Explanation

The drainage from the entire watershed is analyzed to determine the amount of runoff emanating from the area. From the summary tables of the output, it is determined that, for a 12-hour 100-year storm, the peak discharge is 98.45 cfs which occurs after 8.89 hours. Each subbasin can be inspected to determine the amount it contributes to the overall runoff from the watershed. The reach had little or no effect on the time of peak or the discharge. The channel would have to be much wider and longer for any attenuation to be realized.

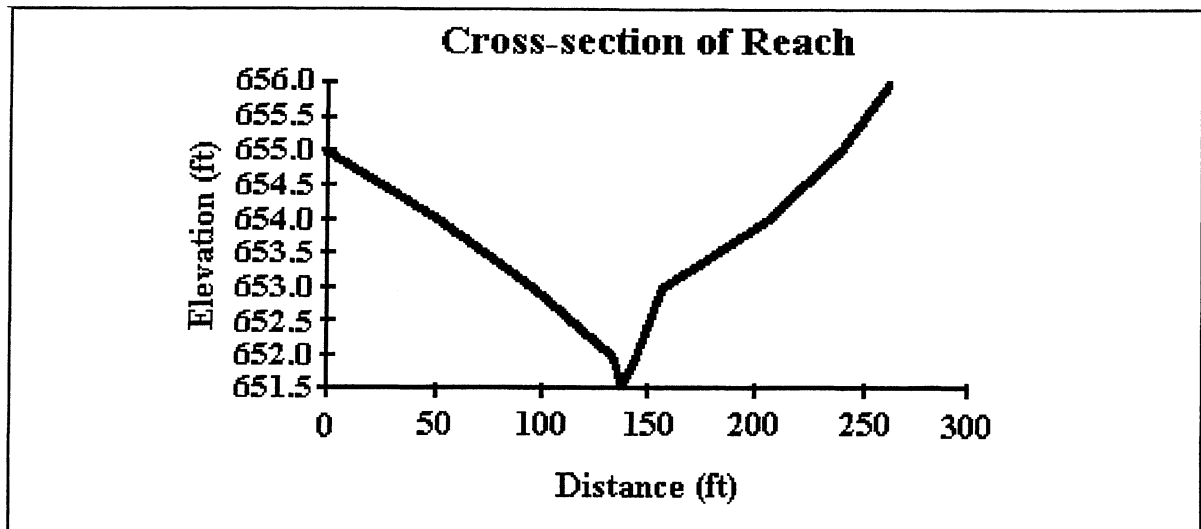


Figure 8.1.6
Cross-section of Reach through Subbasin 2

Table 8.1.8
Summary of Results for Example 8.1.3 for the 12-Hour 100-Year Event

<i>Area</i>	<i>Area (mi²)</i>	<i>Time of Peak (hours)</i>	<i>Peak Discharge</i>
Subbasin 1	0.72	8.44	391.13
Culverts (outlet)	---	12.93	65.47
Reach	---	13.28	65.40
Subbasin 2	0.15	8.39	71.64
Outlet	0.87	8.89	98.45

```

JOB TR-20
TITLE EXAMPLE 8.1.3 - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
TITLE EXISTING CONDITIONS
5 RAINFL 8 .05
8 0.0 .03 .06 .09 .12 HUFF 3RD
8 .15 .19 .23 .27 .32 QUARTILE
8 .38 .45 .57 .70 .79 DISTRIB.
8 .85 .89 .92 .95 .97
8 1. 1. 1. 1. 1.
9 ENDTBL
3 STRUCT 01
8 654.17 0.0 67.5 DEPRESS-
8 654.75 5.0 98.1 IONAL
8 655.08 10.0 117.2 AREA
8 656.10 15.0 195.6
8 656.16 20.0 201.5
8 656.29 30.0 214.2
8 656.43 40.0 228.0
8 656.59 50.0 243.7
9 ENDTBL
2 XSECTN 001 1.0 653. ROSS
8 651.6 0. 0. DITCH
8 652.5 10. 14.0
8 652.7 20. 24.5
8 652.9 40. 35.8
8 653.3 80. 62.4
8 653.5 100. 76.9
8 653.6 150. 97.7
8 653.9 200. 135.8
8 654.0 250. 144.2
8 654.2 300. 176.0
8 654.6 400. 257.7
9 ENDTBL
6 RUNOFF 1 1 1 0.72 84. 1.50 1 0 0 1 0 1 SA 1
6 RESVOR 2 01 1 2 651. 1 0 0 1 0 1 DEPRESS
6 REACH 3 1 2 3 1500. 1 0 0 1 0 1 DITCH
6 RUNOFF 1 2 4 0.15 77. 1.33 1 0 0 1 0 1 SA 2
6 ADDHYD 4 3 3 4 5 1 0 0 1 0 1 @ SR26
ENDATA
7 INCREM 6 0.25
7 COMPUT 7 1 3 0.00 3.91 12. 8 2 1 1 10-YR
ENDCMP 1
7 COMPUT 7 1 3 0.00 5.48 12. 8 2 1 2 100-YR
ENDCMP 1
7 COMPUT 7 1 3 0.00 4.58 24. 8 2 2 1 10-YR
ENDCMP 1
7 COMPUT 7 1 3 0.00 6.41 24. 8 2 2 2 100-YR
ENDCMP 1
ENDJOB 2

```

Figure 8 1 7
TR-20 Input for Example 8 1 3

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
 (A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
 A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION			RUNOFF AMOUNT (IN)	PEAK DISCHARGE				
						BEGIN (HR)	AMOUNT (IN)	DURATION (HR)		ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSH)	
ALTERNATE 1 STORM 1														
+														
XSECTION	1	RUNOFF	.72	8	2	.25	.0	3.91	12.00	2.29	---	8.49	250.20	347.5
STRUCTURE	1	RESVOR	.72	8	2	.25	.0	3.91	12.00	1.41	655.50	13.78	12.05	16.7
XSECTION	1	REACH	.72	8	2	.25	.0	3.91	12.00	1.40	652.54	14.50	12.03	16.7
XSECTION	2	RUNOFF	.15	8	2	.25	.0	3.91	12.00	1.74	---	8.45	42.68	284.5
XSECTION	3	ADDHYD	.87	8	2	.25	.0	3.91	12.00	1.46	---	8.57	47.35	54.4
ALTERNATE 1 STORM 2														
+														
XSECTION	1	RUNOFF	.72	8	2	.25	.0	5.48	12.00	3.71	---	8.44	391.13	543.2
STRUCTURE	1	RESVOR	.72	8	2	.25	.0	5.48	12.00	1.95	656.15	13.73	19.56	27.2
XSECTION	1	REACH	.72	8	2	.25	.0	5.48	12.00	1.94	652.69	14.36	19.34	26.9
XSECTION	2	RUNOFF	.15	8	2	.25	.0	5.48	12.00	3.03	---	8.39	71.64	477.6
XSECTION	3	ADDHYD	.87	8	2	.25	.0	5.48	12.00	2.13	---	8.45	80.82	92.9
ALTERNATE 2 STORM 1														
+														
XSECTION	1	RUNOFF	.72	8	2	.25	.0	4.58	24.00	2.89	---	15.96	177.12	246.0
STRUCTURE	1	RESVOR	.72	8	2	.25	.0	4.58	24.00	1.53	655.71	25.45	13.11	18.2
XSECTION	1	REACH	.72	8	2	.25	.0	4.58	24.00	1.52	652.56	26.08	13.10	18.2
XSECTION	2	RUNOFF	.15	8	2	.25	.0	4.58	24.00	2.27	---	15.93	31.07	207.1
XSECTION	3	ADDHYD	.87	8	2	.25	.0	4.58	24.00	1.65	---	16.10	39.37	45.3
ALTERNATE 2 STORM 2														
+														
XSECTION	1	RUNOFF	.72	8	2	.25	.0	6.41	24.00	4.58	---	15.93	270.09	375.1
STRUCTURE	1	RESVOR	.72	8	2	.25	.0	6.41	24.00	2.32	656.35	25.07	34.05	47.3
XSECTION	1	REACH	.72	8	2	.25	.0	6.41	24.00	2.31	652.84	25.46	33.98	47.2
XSECTION	2	RUNOFF	.15	8	2	.25	.0	6.41	24.00	3.84	---	15.88	50.35	335.7
XSECTION	3	ADDHYD	.87	8	2	.25	.0	6.41	24.00	2.57	---	15.93	61.96	71.2

1

Figure 8.1 8
 TR-20 Output for Example 8.1 3

8 2 - HEC-1 PROGRAM

8.2.1 - Explanation of HEC-1 and its Uses

HEC-1 is a flood hydrograph package developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center (HEC), which is used to estimate direct runoff from precipitation data. The size and complexity of the basins simulated vary from small urban basins to a large river systems. HEC-1 can be used to (Bedient & Huber, 1988):

- 1 Estimate unit hydrographs, loss rates, and streamflow routing parameters from measured data;
- 2 Simulate streamflow from historical or design rainfall data;
- 3 Compute damage frequency curves and expected annual damages for various locations and multiple flood control plans;
4. Simulate reservoir outflows for dam safety analysis.

In addition, the program has several other features which may be used to compute snowmelt simulation, parameter estimation, dam safety analysis, pumping and diversion schemes, multiple-flood and multiple-plan analyses, and simulation of precipitation depth-area relationships.

In this section the program is applied to several cases with emphasis on a couple of the features. Further information on the capabilities of HEC-1 the manual and software can be obtained by writing to The Hydrologic Engineering Center, U.S. Army Corps of Engineers, Attention Training Division, 609 Second Street, Davis, CA 95616. Part of the software that accompanies HEC-1 is a full-screen text-editing program named COED which provides on-line help screens and input variable documentation to prepare data input files. COED is very useful when creating models for the first time and saves a great deal of time spent looking up variables and their parameters.

8 2 2 - Summary of HEC-1 Input Structure

When developing a precipitation-runoff model the boundaries of the basin must be identified initially. Most often the basin is subdivided into smaller subbasins depending on the study objectives, drainage pattern and other factors. It is important to identify points where runoff information is needed. The model can be structured to produce hydrographs at any desired location. As different areas of a large basin may have different hydrologic response characteristics, it is important to select an appropriate computational time interval and subdivide the area so that lumped parameters provide a reasonable depiction of the watershed.

There are several methods in HEC-1 to compute surface runoff. These are: (1) the unit hydrograph input directly, (2) the Clark or the Snyder synthetic unit hydrographs, (3) SCS method (curve number method and SCS unit hydrograph), and (4) kinematic wave for overland hydrograph. Losses may be computed by using one of the five methods shown in Table 8.2.1.

Table 8.2.1
Estimation of Losses in HEC-1 (Bedient & Huber, 1988)

<i>METHOD</i>	<i>DESCRIPTION</i>
Initial and Constant	Initial loss volume is satisfied and then constant loss rate begins.
HEC exponential	Loss function is related to antecedent moisture condition and is a continuous function of soil wetness.
SCS curve number	Initial loss is satisfied before calculating cumulative runoff as a function of cumulative rainfall.
Holtan method	Infiltration rate is computed as an exponential function of available soil moisture storage from Holtan's equation.

Flood routing is used to simulate flood wave movement through river reaches and reservoirs. Most of the flood routing procedures are based on the continuity equation and some relationship between flow and storage. Some of the methods available in HEC-1 are: (1) Muskingum-storage coefficient plus travel time through each reach, (2) normal-depth channel routing - Manning's equation is used to compute a table of storage versus outflow values for use in Modified Puls routing (described below), (3) Kinematic wave -outflow from each reach based on depth of flow in continuity and Manning's equations. There are other methods, not discussed here, which deviate slightly from these principle methods.

The modified Puls method, also known as Storage-Indication, utilized to route reservoirs, consists of a repetitive solution of the continuity equation (Equation 6.2.2) on the basis of assumptions that the reservoir water surface remains horizontal and that outflow is a unique function of storage. The continuity equation can be rearranged to isolate the two unknowns for the first time step, O_2 and S_2 .

$$\frac{S_2}{\Delta t} + \frac{O_2}{2} = \left(\frac{S_1}{\Delta t} + \frac{O_1}{2} \right) - O_1 + \frac{I_1 + I_2}{2} \quad (8.2.1)$$

Since I is known for all time steps, and O_1 and S_1 are known for the first time step, the right hand side of Equation 8.2.1 can be determined. Values of O_2 and S_2 are then used as input on the left-hand side, and the computation is repeated for the next time interval, and so on. A storage-indications curve is a plot of $[(S/\Delta t) + (O/2)]$ vs O .

The procedure for applying the modified Puls method is as follows (Hoggan, 1989):

1. Determine a discharge rating curve for the reservoir outlet.
2. Determine reservoir storage that correlates with each elevation on the rating curve for reservoir outflow.

3. Construct a storage indication vs. outflow curve: $[(S/\Delta t) + (O/2)]$ vs. O .
4. Route the inflow hydrograph through the reservoir on the basis of Equation 8.2.1.
5. Verify the results with data from historical events.

When the modified Puls method is used in HEC-1 to do reservoir routing, the storage and outflow values are required input data. Storage-outflow input data may be specified in two ways (1) as precomputed storage vs. elevation or discharge data and (2) as water surface area vs elevation data

A "cascade of reservoirs" may be used to depict storage routing in a river. The river is divided into a series of routing reaches represented by reservoirs, with the outflow from the first reservoir being the inflow to the second, the outflow from the second being the inflow to the third, and so on. Caution must be used when applying the modified Puls method to channel routing. The degree of attenuation introduced in the routed flood wave varies depending on the river reach lengths chosen, or alternatively, on the number of routing steps specified for a single reach

When this method is applied to a reach, a hydrograph at an upstream location is routed to a downstream location. The storage is the volume in the channel under the water surface profile, and the outflow is the discharge in the channel at the downstream end of the reach. A storage-outflow relationship must be defined which is similar to that for reservoir routing, but the approach is different. In river reaches, storage-outflow relationships are determined from one of the following: (1) steady-flow profile computations, (2) observed profiles, (3) normal-depth calculations, (4) storage calculations based on inflow and outflow hydrographs, and (5) optimization techniques applied to inflow and outflow hydrographs.

1 Input Data Structure

In the HEC-1 model "cards" of input which indicate the format for the river basin data and determine output formats are used. A "card" refers to an input line of 80 columns in either fixed or free format. Fixed format consists of ten 8-column fields and free format allows data values to be separated by commas. For the fixed format, if a decimal point is not used, all numbers must be right justified within the field. Blank fields are read as zeros.

There are two identification characters in columns one and two which form a unique two-character classification system. The first character identifies a data category, and the second indicates a type of data or option within the category. For example, an "R" in the first column indicates routing method, which has seven subcategories, as indicated by the second character in the line identifiers - RC, RK, RL, RM, RN, RS and RT. For example, RK is used to indicate the kinematic wave channel routing method. A list of the basic data categories is shown in Table 8.2.2. For detailed information on different lines of input and their methods, one should refer to Appendix A of the HEC-1 users manual (HEC-1, 1990). Line input descriptions and variable definitions are provided in help files of the text editor COED.

Table 8 2.2
Basic HEC-1 Data Categories

Job Control	
I_	Job initialization
V_	Variable Output summary
O_	Optimization
J_	Job type
<i>Hydrology and Hydraulics</i>	
K_	Job step control
H_	Hydrograph transformation
Q_	Hydrograph data
B_	Basin data
P_	Precipitation data
L_	Loss (infiltration) data
U_	Unitgraph data
M_	Melt data
R_	Routing data
S_	Storage data
D_	Diversion data
W_	Pump withdrawal data
<i>Economics and end of job</i>	
E_	Economics data
ZZ	End of job

The sequence of lines for an input file follows a specific order which describes the hydrologic flow and routing. The job control information comes first, followed by the hydrology and hydraulics information, and finally the economic information. The input file always begins with an ID for the first record. Other job control data which follow, describe the job type, output control, time interval, time span, and the type of units selected (where the default is

english units). The hydrology and hydraulics data include all the data necessary to simulate various physical processes in the river basin. The KK record is used at the beginning of each hydrograph calculation, followed by other lines providing data to compute the hydrograph. For example, subbasin runoff, hydrograph combination, and a routing step begin with a KK line, giving the input a "block" structure. The economic data begins with an EC line and is used to compute the expected annual damages. A ZZ record is the last line in an input file, signaling the end of the job. An example data organization for a watershed is shown in Table 8.2.4 (Bedient & Huber, 1988).

Time data are entered on IT and IN lines. A time interval and a time span are used to calculate runoff hydrographs, and these are defined with variables on the IT line. Time-series data are input with different time parameters than those used on the simulation. For example, the precipitation data can have different starting times and dates and a different time interval. These are specified on the IN line is located in the job right before the precipitation data. The precipitation data are interpolated to fit the simulation time conditions that have been specified on the IT line. The precipitation data may be input as basin average rainfall, gage data, synthetic storm data, incremental or cumulative rainfall as a time series, probable maximum storm, etc (Hoggan, 1989). If the precipitation does not change from one subbasin to another, then the data may be input only once, with the first KK group. The different precipitation input options are shown in Table 8.2.3.

Table 8.2.3
Precipitation Input Options

<i>Line Identification</i>	<i>Type of Storm</i>
PI or PC	Incremental or cumulative rainfall as a time series
PB	Basin average precipitation
PG	Total rainfall for a nonrecording gage or to indicate that recorded data follow on PI or PC lines
PR, PW and PT, PW	Specify which gages and corresponding weights are to be used to calculate a subbasin's average precipitation
PH	Hypothetical storm from depth-duration data
PM	Probable maximum storm
PS	Standard project storm

Table 8.2.4
Example of Input Data Organization

<i>Segment</i>	<i>Code</i>	<i>Description of Data</i>
Job control	ID	Title and description of model
	IT/IN	Time interval and beginning time
	IO	Output control for entire job
Runoff from first subbasin	KK	Station name
	KM	Comment card (optional)
	BA	Basin area
	P_	Precipitation method
	L_	Loss rate method
	U_	Rainfall transformation method
Route hydrograph from subbasin to next point	KK	Station name
	KM	Comment card (optional)
	R_	Routing method
Runoff from next subbasin	KK	Station name
	KM	Comment card (optional)
Combine hydrographs	KK	Station name
	KM	Comment card (optional)
	HC	Number of hydrographs to be combined
Process continues until mouth of the watershed is reached		
	ZZ	Indicates end of job

After a storm hydrograph has been computed, channel and reservoir routing are performed using methods indicated by R_ lines. For storage-discharge relations data is input on the SV/SQ lines, and the SQ/SE lines are used to input a rating curve of discharge versus elevation. For reservoir routing, a number of lines can be used to fully model outflow structures. The SA, SV, and SE lines describe reservoir area-volume-elevation tables, and SS or SL lines specify spillway and low-level outlet data.

Two or more hydrographs are summed by the HC combination line. The HC step uses the most recently calculated hydrographs in the system. The progression of hydrologic calculation follows the order of the KK blocks. If a gage exists at any point to measure the flows, the QO line can be used to compare computed and observed flows.

There are other options and features that HEC-1 can perform. The previous description covers the basics for the modeling of a watershed. The following examples are used to demonstrate some of the features of the program.

8 2.3 - Example Problems

Example 8.2.1

1 Description

The runoff from Subbasin 1 (SA1) of the watershed shown on Exhibit 8.2.1 is estimated using the SCS method in HEC-1. The area of the subbasin is 461 acres or 0.72 sq mi.. The curve number and the time of concentration for this area are determined to be 84 and 1.50 hours, respectively. The precipitation data for the subbasin is a 12-hour 100-year event. From Chapter 2, for a 100-year 12-hour event for the Lafayette area, the rainfall depth is 5.48 inches. This is distributed over the subbasin incrementally.

2 Computer Input

Figure 8 2 1 shows the input data. The input is the fixed format, where the data is input to specified "fields." A "field" refers to 8 column spaces and they are numbered sequentially starting with columns 3 through 10.

ID - The first two lines are used to describe the location and conditions, as well as other descriptions. These comments are written in columns 3 through 80, after an ID is placed as the identifier.

IT - Following the identification is an IT line which specifies the time. The number of minutes in the tabulation interval is 36, which is placed in field 1 of the IT line. The 36 minutes is determined by dividing the storm duration, in minutes (720 minutes), by the number of increments in the precipitation data. It is possible to give the first time interval and beginning time. For example, if it is desired to simulate a storm starting at 2:45 p.m. on July 14, 1989, the input would be '14JUL89' in field 2 and '1445' in field 3. Field 4 is used to give the number of hydrograph ordinates to be computed, specified here as 50 (300 maximum). Fields 5 and 6 give the ending date and time, respectively, if fields 2 and 3 were used and then the number of points in the hydrograph would be computed automatically.

```

ID  EXAMPLE 8.2.1
ID  SUBBASIN 1 - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
ID  EXISTING CONDITIONS - 12-HOUR 100-YEAR EVENT
IT   36      0      0      50
IO   5      2
KK   SA1
KM  HYDROGRAPH FOR SUBBASIN 1
BA   .72
PB  5.48
PC   0.0      0.03      0.06      0.09      0.12      0.15      0.19      0.23      0.27      0.32
PC   0.38      0.45      0.57      0.70      0.79      0.85      0.89      0.92      0.95      0.97
PC   1.0
LS   0      84
UD   0.90
ZZ

```

Figure 8 2 1
HEC-1 Input Data for Example 8 2 1

IO - The IO line is used to control the output. The '5' in the first field, enables printing of the job specification and a master summary. A '2' in the second field indicates that plots of all hydrographs will be made ('0' suppresses plots). The KO line is used to change output control for each hydrograph calculation.

KK - A hydrograph will be calculated following the data after the KK "block" of lines. The station location identification is placed in field 1. It must be a unique identifier for the entire run.

KM - The message in the KM line is printed at the beginning of the output for each station or plan.

BA - The drainage area in square miles is placed in field 1. The area of subbasin 1 is 461 acres or 0.72 sq. mi.

PB - This is the basin average precipitation line. The total storm, basin average precipitation in inches is 5.48, which is placed in field 1.

PC - The cumulative precipitation distribution is given on the PC lines. The PB line specified the amount of precipitation and the previous IT line specified interval length, which was 36 minutes. If the time interval and starting time is different for this subbasin than what was specified on the IT line, then an IN line can be put immediately before this to specify the times for this hydrograph. There are 20 incremental precipitation values for this time distribution,

each at 36 minute intervals. The precipitation values sum to 1.0 and when multiplied by the total storm give the basin average precipitation of 5.48 inches over 12 hours. Because there are only ten fields available per line, three PC lines are used. Recall that the first value, 0.0 is not counted

LS - The SCS curve number loss rate is specified here. Since a '0' was placed in the first field, the initial abstraction will be computed as.

$$I_a = 0.2 \frac{1000 - 10 * CN}{CN}$$

Field 2 specifies the SCS curve number which is 84. The third field may be used to give the percent of the drained basin that is impervious

UD - Used to specify the SCS lag in hours. The lag is 0.6 times the t_c or 0.90 hrs.

ZZ - Indicates the end of a job

3 Computer Output

The form of the output is controlled by the IO line. In this example the input and a master summary was requested by placing a '5' in the first field and a plot of the hydrograph from Subbasin 1 was requested by the '2' in the second field. Figure 8.2.2 contains the full output file.

4 Summary/Explanation

The results from Example 8.2.1 show that the peak flow for the present conditions from Subbasin 1 is 379 cfs. The time to peak is 8.40 hrs. These values are slightly less than the values obtained from TR-20. The difference is explained by the method used to compute the peak runoff. Although both methods use the SCS Hydrograph method, HEC-1 uses the triangular unit hydrograph and TR-20 uses the dimensionless unit hydrograph to compute the peak runoff.

```

*****
*
* FLOOD HYDROGRAPH PACKAGE (HEC-1)
* SEPTEMBER 1990
* VERSION 4.0
*
*****

```

```

*****
*
* U.S. ARMY CORPS OF ENGINEERS
* HYDROLOGIC ENGINEERING CENTER
* 609 SECOND STREET
* DAVIS, CALIFORNIA 95616
*
*****

```

```

X X XXXXXX XXXX X
X X X X X XX
X X X X X
XXXXXX XXXX X XXXX X
X X X X X
X X X X X
X X XXXXXX XXXX XXX

```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
 THE DEFINITION OF -AMSK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
 NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
 DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
 KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

HEC-1 INPUT

PAGE 1

LINE	ID.....1.....2.....3.....4.....5.....6.....7.....8.....9.....10
1	ID EXAMPLE 8.2.1
2	ID SUBBASIN 1 - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
3	ID EXISTING CONDITIONS - 12-HOUR 100-YEAR EVENT
4	IT 36 0 0 50
5	IO 5 2
6	KK SA1
7	KM HYDROGRAPH FOR SUBBASIN 1
8	BA .72
9	PB 5.48
10	PC 0.0 0.03 0.06 0.09 0.12 0.15 0.19 0.23 0.27 0.32
11	PC 0.38 0.45 0.57 0.70 0.79 0.85 0.89 0.92 0.95 0.97
12	PC 1.0
13	LS 0 84
14	UD 0.90
15	ZZ

```

*****
*
* FLOOD HYDROGRAPH PACKAGE (HEC-1)
* SEPTEMBER 1990
* VERSION 4.0
*
*****

```

```

*****
*
* U.S. ARMY CORPS OF ENGINEERS
* HYDROLOGIC ENGINEERING CENTER
* 609 SECOND STREET
* DAVIS, CALIFORNIA 95616
*
*****

```

Figure 8 2.2(a)
HEC-1 Output from Example 8.2 1

EXAMPLE 8.2.1
SUBBASIN 1 - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
EXISTING CONDITIONS - 12-HOUR 100-YEAR EVENT

5 IO OUTPUT CONTROL VARIABLES
 IPRNT 5 PRINT CONTROL
 IPLOT 2 PLOT CONTROL
 QSCAL 0. HYDROGRAPH PLOT SCALE

IT HYDROGRAPH TIME DATA
 NMIN 36 MINUTES IN COMPUTATION INTERVAL
 IDATE 1 0 STARTING DATE
 ITIME 0000 STARTING TIME
 NQ 50 NUMBER OF HYDROGRAPH ORDINATES
 NDDATE 2 0 ENDING DATE
 NDTIME 0524 ENDING TIME
 ICENT 19 CENTURY MARK

 COMPUTATION INTERVAL .60 HOURS
 TOTAL TIME BASE 29.40 HOURS

ENGLISH UNITS
DRAINAGE AREA SQUARE MILES
PRECIPITATION DEPTH INCHES
LENGTH, ELEVATION FEET
FLOW CUBIC FEET PER SECOND
STORAGE VOLUME ACRE-FeET
SURFACE AREA ACRES
TEMPERATURE DEGREES FAHRENHEIT

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD			BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
					6-HOUR	24-HOUR	72-HOUR			
+	HYDROGRAPH AT									
+		SA1	379.	8.40	232.	72.	59.	.72		

*** NORMAL END OF HEC-1 ***

Figure 8.2 2(b)
HEC-1 Output from Example 8 2 1

		STATION								SA1			
		(O) OUTFLOW											
		0.	50.	100.	150.	200.	250.	300.	350.	400.	0.	0.	0.
										(L) PRECIP, (X) EXCESS			
		.0	.0	.0	.0	.0	.0	.0	.0	.8	.6	.4	.2
DAHRMN PER													
10000	10	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
10036	20	LLLLLLLL.
10112	30	LLLLLLLL.
10148	40	LLLLLLLL.
10224	5.0	LLLLLLXY.
10300	6. 0	LLLLLLXY.
10336	7. 0	LLLLLLXXXX.
10412	8. 0	LLLLLLXXXX.
10448	9. 0	LLLLLLXXXX.
10524	10. 0	LLLLXXXXXXXX.
10600	11. 0	LLLLXXXXXXXX.
10636	12. 0	LLLLXXXXXXXX.
10712	13. 0	LLLLXXXXXXXX.
10748	14. 0	LLLLXXXXXXXX.
10824	15. 0	LLLLXXXXXXXX.
10900	16. 0	LLLLXXXXXXXX.
10936	17. 0	LLLLXXXXXXXX.
11012	18. 0	XXXXXXXY.
11048	19. 0	XXXXXXXY.
11124	20. 0	XXXXXXXY.
11200	21. 0	XXXXXXXY.
11236	22. 0	XXXXXXXY.
11312	23. 0	XXXXXXXY.
11348	24. 0	XXXXXXXY.
11424	25. 0	XXXXXXXY.
11500	26.0	XXXXXXXY.
11536	270	XXXXXXXY.
11612	280	XXXXXXXY.
11648	290	XXXXXXXY.
11724	300	XXXXXXXY.
11800	310	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
11836	320	XXXXXXXY.
11912	330	XXXXXXXY.
11948	340	XXXXXXXY.
12024	350	XXXXXXXY.
12100	360	XXXXXXXY.
12136	370	XXXXXXXY.
12212	380	XXXXXXXY.
12248	390	XXXXXXXY.
12324	400	XXXXXXXY.
20000	410	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
20036	420	XXXXXXXY.
20112	430	XXXXXXXY.
20148	440	XXXXXXXY.
20224	450	XXXXXXXY.
20300	460	XXXXXXXY.
20336	470	XXXXXXXY.
20412	480	XXXXXXXY.
20448	490	XXXXXXXY.
20524	500	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----

Figure 8 2 2(c)
HEC-1 Output for Example 8.2 1

Example 8.2.2

1 Description

The flow from Subbasin 1 is routed through twin 38" by 60" culverts under Interstate 65.

2 Calculations

A stage-storage-discharge relationship is calculated for the culverts. This is shown in Table 8.1.4. The curve number and time of concentration for this example are the same as the previous example

3 Computer Input

Using the input from the previous example, another hydrograph is added for the computation of the routing through the culverts, so another KK record is used. The culverts elevation-storage-discharge relationship is input with the SV (for storage), SE (for elevation) and SQ (for discharge). The RS record is required for storage routing. The ELEV and 654.17 in the RS record indicates the starting elevation of the pond. The '1' indicates the number of steps used in the storage routing. These are the only additions needed. The input file for this example is shown in Figure 8.2.3

4 Output

The output is in the same format as Example 8.2.1 since the IO record is not changed. The output is given in Figure 8.2.4. In order to decrease the size of this chapter, only a brief summary of the results is presented. Most of the original output has been deleted.

5 Summary/Explanation

The flowrate from the culverts is nearly one twentieth of the flow from subbasin 1 because of the detention through the depressional area. The peak outflow from the culverts is 19 cfs at a peak time of 13.80 hrs. The culverts are large enough to pass the detained flow without overtopping. The peak elevation is 656.15 feet. These are the same results found using TR-20.


```

ID  EXAMPLE 8.2.2
ID  SUBBASIN 1 AND CULVERTS - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
ID  EXISTING CONDITIONS - 12-HOUR 100-YEAR EVENT
IT  36      0      0      50
IO  5      0
KK  SA1
KM  HYDROGRAPH FOR SUBBASIN 1
BA  .72
PB  5.48
PC  0.0    0.03   0.06   0.09   0.12   0.15   0.19   0.23   0.27   0.32
PC  0.38   0.45   0.57   0.70   0.79   0.85   0.89   0.92   0.95   0.97
PC  1.0
LS  0      84
UD  0.90
KK  DEPR
KM  ROUTE SUBBASIN 1 HYDROGRAPH THROUGH DEPRESSIONAL AREA
RS  1      ELEV  654.17
SV  67.5   98.1   117.2   195.6   201.5   214.2   228.0   243.7
SE 654.17  654.75  655.08  656.10  656.16  656.29  656.43  656.59
SQ  0      5.0    10.0    15.0    20.0    30.0    40.0    50.0
ZZ

```

Figure 8 2.3
HEC-1 Input for Example 8.2 2

EXAMPLE 8.2.2
SUBBASIN 1 AND CULVERTS - ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
EXISTING CONDITIONS - 12-HOUR 100-YEAR EVENT

```

5 IO      OUTPUT CONTROL VARIABLES
          IPRNT      5  PRINT CONTROL
          IPLOT      0  PLOT CONTROL
          QSCAL      0.  HYDROGRAPH PLOT SCALE

IT        HYDROGRAPH TIME DATA
          NMIN       36  MINUTES IN COMPUTATION INTERVAL
          IDATE      1  0  STARTING DATE
          ITIME      0000 STARTING TIME
          NQ         50  NUMBER OF HYDROGRAPH ORDINATES
          NDDATE     2  0  ENDING DATE
          NDTIME     0524 ENDING TIME
          ICENT      19  CENTURY MARK

          COMPUTATION INTERVAL .60 HOURS
          TOTAL TIME BASE 29.40 HOURS

ENGLISH UNITS
DRAINAGE AREA      SQUARE MILES
PRECIPITATION DEPTH INCHES
LENGTH, ELEVATION  FEET
FLOW               CUBIC FEET PER SECOND
STORAGE VOLUME     ACRE-FEET
SURFACE AREA       ACRES
TEMPERATURE        DEGREES FAHRENHEIT

1

                                RUNOFF SUMMARY
                                FLOW IN CUBIC FEET PER SECOND
                                TIME IN HOURS, AREA IN SQUARE MILES

OPERATION      STATION      PEAK    TIME OF    AVERAGE FLOW FOR MAXIMUM PERIOD
+              +          FLOW    PEAK      6-HOUR    24-HOUR    72-HOUR
+              +          +      +
HYDROGRAPH AT
+              SA1        379.    8.40      232.      72.      59.      .72
ROUTED TO
+              DEPR       19.     13.80     18.       14.      11.      .72
+
+                                     656.15    13.80

*** NORMAL END OF HEC-1 ***

```

Figure 8 2 4
HEC-1 Output for Example 8 2.2

Example 8.2.3

1 Description

The entire watershed is analyzed to compute the peak runoff hydrograph from the watershed area of 0.85 square miles. This example incorporates the same area as Example 8.2.2 and routes the outflow from the culverts through a ditch and adds the runoff from subbasin 2 to determine the peak runoff at the outlet of the watershed, State Road 26. The stage-storage-discharge relationship for the ditch is determined from Figure 8.1.6.

2. Calculations

The curve number and time of concentration are shown in Tables 8.1.6 and 8.1.7 for Subbasin 2. The area is calculated in square miles. All of these calculations are explained in Section 8.2.2. For the routing of the outflow from the culverts through the ditch to the discharge point of the basin, it is not necessary to calculate the stage-storage-discharge relation, as it was in TR-20.

3 Computer Input

An additional hydrograph is input for Subbasin 2, a routing hydrograph for the ditch, and a combine hydrograph for adding hydrographs together. As with the other example problems for HEC-1, the KK record is used before any station computation, subbasin runoff, routing, etc.

The routing input is very simple. There are 3 records (RC, RX and RY) for describing the channel. In the RC record, the values for the channel's Manning's n , length and slope are input. The RX and RY records define the cross-section of the channel as shown in Figure 8.1.6. The stage-storage-discharge relation is internally computed from this information. The HC record is used for combining subbasin 2 and the hydrograph from the ditch routing. The input file for this example is shown in Figure 8.2.5.

4 Output

The output is shown on Figure 8.2.6. For the final example, the input, hydrographs and summary table are shown.

5 Summary/Explanation

The peak outflow from the watershed is 78 cfs and the time to peak is 8.40 hrs. As with the TR-20 example, the routing of the flow through the ditch to the outlet has insignificant effects on the time to peak. This outflow value is close to the value obtained from TR-20 which has a peak flow value of 80.8 cfs.

```

ID  EXAMPLE 8.2.3
ID  ROSS DITCH WATERSHED - TIPPECANOE COUNTY, IN
ID  EXISTING CONDITIONS - 12-HOUR 100-YEAR EVENT
IT  36      0      0      50
IO  3      0
KK  SA1
KM  HYDROGRAPH FOR SUBBASIN 1
BA  .72
PB  5.48
PC  0.0      0.03      0.06      0.09      0.12      0.15      0.19      0.23      0.27      0.32
PC  0.38      0.45      0.57      0.70      0.79      0.85      0.89      0.92      0.95      0.97
PC  1.0
LS  0      84
UD  0.90
KK  DEPR
KM  ROUTE SUBBASIN 1 HYDROGRAPH THROUGH DEPRESSIONAL AREA
RS  1      ELEV  654.17
SV  67.5      98.1      117.2      195.6      201.5      214.2      228.0      243.7
SE 654.17      654.75      655.08      656.10      656.16      656.29      656.43      656.59
SQ  0      5.0      10.0      15.0      20.0      30.0      40.0      50.0
KK  DITCH
KM  HYDROGRAPH FROM DEPRESSIONAL AREA IS ROUTED THROUGH DITCH
RD
RC  .09      .08      .09      1500.      .0065
RX  0.      52.      97.      140.      145.      205.      239.      264.
RY  655.      654.      653.      651.6      652.      654.      655.      656.
KK  SA2
KM  HYDROGRAPH FOR SUBBASIN 2
BA  .15
LS  0      77
UD  .798
KK  ROAD
KM  COMBINE DITCH OUTFLOW AND SUBBASIN 2 AT STATE ROAD 26
KO  0      2
HC  2
ZZ

```

Figure 8 2 5
HEC-1 Input for Example 8.2.3

```

*****
21 KK  *   DITCH   *
*****
        HYDROGRAPH FROM DEPRESSIONAL AREA IS ROUTED THROUGH DITCH

```

HYDROGRAPH ROUTING DATA

```

23 RD      MUSKINGUM-CUNGE CHANNEL ROUTING

```

```

24 RC      NORMAL DEPTH CHANNEL
          ANL      .090  LEFT OVBANK N-VALUE
          ANCH     .080  MAIN CHANNEL N-VALUE
          ANR      .090  RIGHT OVBANK N-VALUE
          RLNTH    1500. REACH LENGTH
          SEL      .0065 ENERGY SLOPE
          ELMAX     .0    MAX. ELEV. FOR STORAGE/OUTFLOW CALCULATION

```

CROSS-SECTION DATA

```

          --- LEFT OVBANK --- + ----- MAIN CHANNEL ----- + --- RIGHT OVBANK ---
26 RY      ELEVATION  655.00  654.00  653.00  651.60  652.00  654.00  655.00  656.00
25 RX      DISTANCE   .00    52.00   97.00  140.00  145.00  205.00  239.00  264.00

```

COMPUTED STORAGE-OUTFLOW-ELEVATION DATA

STORAGE	.00	.04	.16	.39	.72	1.17	1.73	2.42	3.24	4.20
OUTFLOW	.00	.41	2.56	7.82	17.80	33.70	56.62	90.87	134.50	188.56
ELEVATION	651.60	651.83	652.06	652.29	652.53	652.76	652.99	653.22	653.45	653.68
STORAGE	5.30	6.54	7.95	9.50	11.22	13.09	15.04	17.03	19.06	21.14
OUTFLOW	254.11	339.21	442.80	562.24	698.58	856.20	1039.23	1240.01	1458.24	1693.71
ELEVATION	653.92	654.15	654.38	654.61	654.84	655.07	655.31	655.54	655.77	656.00

COMPUTED MUSKINGUM-CUNGE PARAMETERS

ELEMENT	ALPHA	N	COMPUTATION TIME STEP		PEAK	TIME TO PEAK	VOLUME	MAXIMUM CELERITY
			DT	DX				
			(MIN)	(FT)				
MAIN			29.52	750.00	19.43	856.09	.70	.85

INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

MAIN	36.00	19.35	864.00	.71
------	-------	-------	--------	-----

CONTINUITY SUMMARY (AC-FT) - INFLOW= .2773E+02 EXCESS= .0000E+00 OUTFLOW= .2703E+02 BASIN STORAGE= .6004E+00 PERCENT ERROR= .3

```

***          ***          ***          ***          ***
          HYDROGRAPH AT STATION      DITCH

PEAK FLOW      TIME      MAXIMUM AVERAGE FLOW
+ (CFS)      (HR)      6-HR      24-HR      72-HR      29.40-HR
+ 19.      14.40      (CFS)
          (INCHES)      .226      .707      .708      .708
          (AC-FT)      9.      27.      27.      27.
          CUMULATIVE AREA =      .72 SQ MI

```

Figure 8 2.6(a)
HEC-1 Output for Example Problem 8.2.3

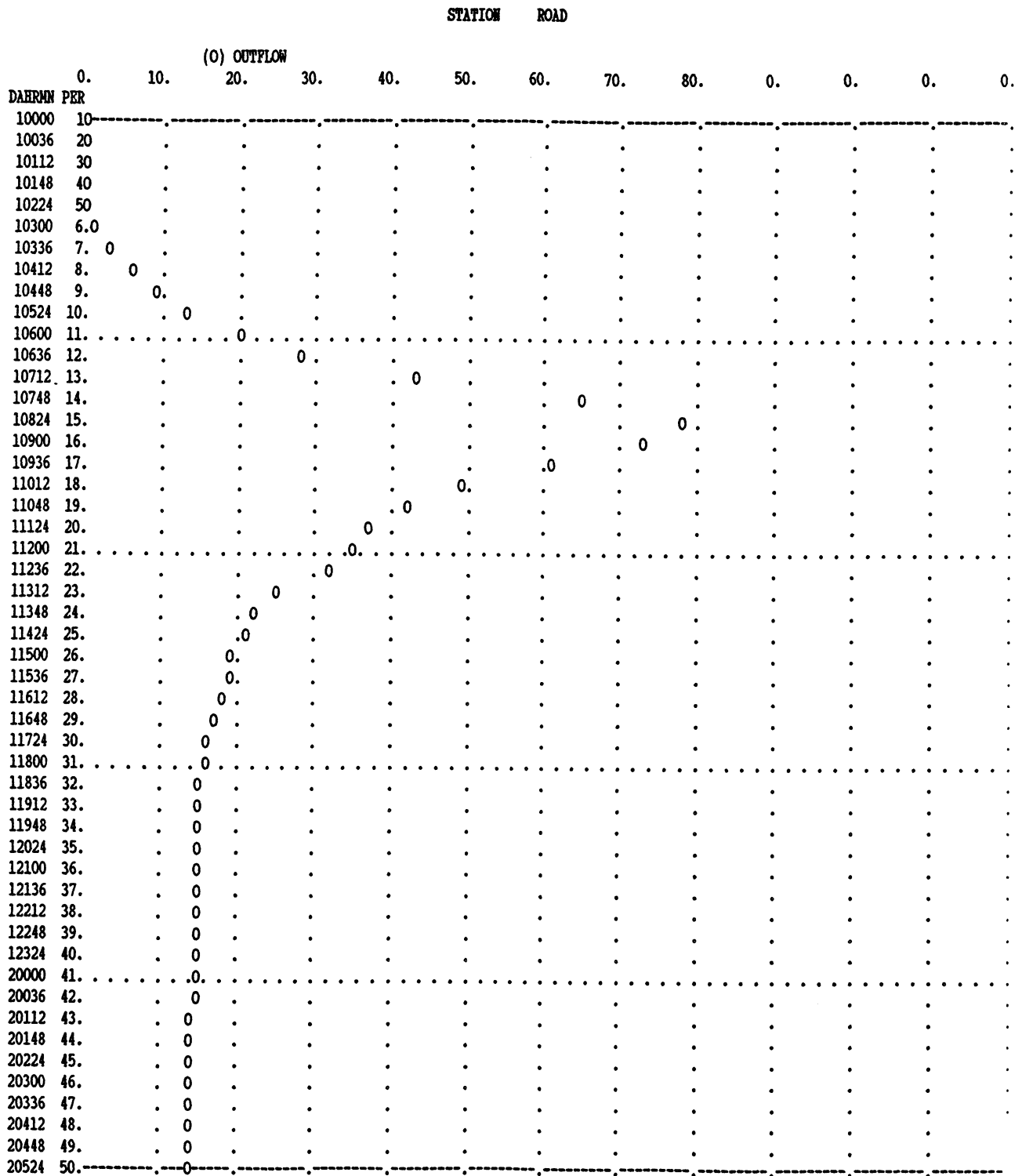


Figure 8.2.6(b)
HEC-1 Output for Example Problem 8 2 3

RUNOFF SUMMARY										
FLOW IN CUBIC FEET PER SECOND										
TIME IN HOURS, AREA IN SQUARE MILES										
	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD			BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+					6-HOUR	24-HOUR	72-HOUR			
	HYDROGRAPH AT									
+		SA1	379.	8.40	232.	72.	59.	.72		
	ROUTED TO									
+		DEPR	19.	13.80	18.	14.	11.	.72		
+									656.15	13.80
	ROUTED TO									
+		DITCH	19.	14.40	18.	14.	11.	.72		
	HYDROGRAPH AT									
+		SA2	69.	8.40	41.	12.	10.	.15		
	2 COMBINED AT									
+		ROAD	78.	8.40	51.	25.	21.	.87		
1										

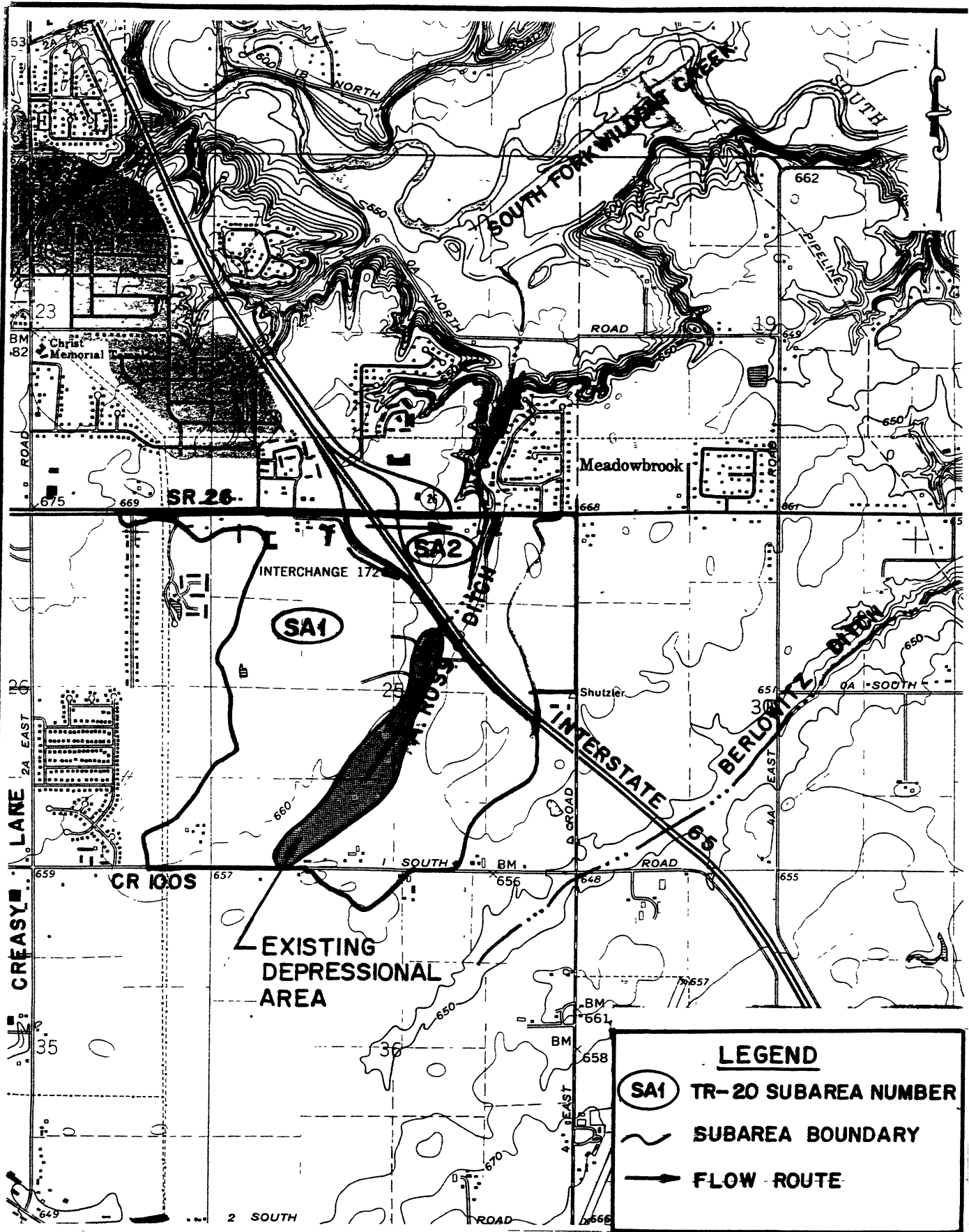
SUMMARY OF KINEMATIC WAVE - MUSKINGUM-CUNGE ROUTING
(FLOW IS DIRECT RUNOFF WITHOUT BASE FLOW)

ISTAQ	ELEMENT	DT	PEAK	TIME TO PEAK	VOLUME	DT	INTERPOLATED TO COMPUTATION INTERVAL		VOLUME
							PEAK	TIME TO PEAK	
							(CFS)	(MIN)	
		(MIN)	(CFS)	(MIN)	(IN)	(MIN)	(CFS)	(MIN)	(IN)
	DITCH MANE	29.52	19.43	856.09	.70	36.00	19.35	864.00	.71

CONTINUITY SUMMARY (AC-FT) - INFLOW= .2773E+02 EXCESS= .0000E+00 OUTFLOW= .2703E+02 BASIN STORAGE= .6004E+00 PERCENT ERROR= .3

*** NORMAL END OF HEC-1 ***

Figure 8 2.6(c)
HEC-1 Output for Example Problem 8 2 3



NOTE: BASE MAP OBTAINED FROM USGS LAFAYETTE EAST, IN. QUADRANGLE

SCALE: 1"=2000'



CHRISTOPHER B. BURKE ENGINEERING, LTD
10275 WEST HIGGINS ROAD SUITE 460
ROSEMONT, ILLINOIS 60018 (708) 296-0500

CLIENT
TIPPECANOE COUNTY

PROJECT No

TITLE

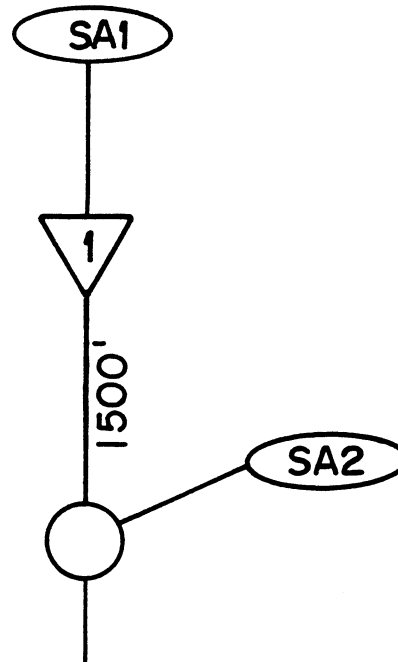
**EXISTING CONDITION WATERSHED
DELINEATION MAP**

APPROVED

DATE

EXHIBIT 8.2.1

A. ROSS DITCH
EXISTING CONDITION
TR-20 MODEL
FLOW SCHEMATIC



LEGEND:

(SA1) TR-20 SUBAREA

○ COMBINE FLOWS

1500' REACH ROUTING

▽ 1 RESERVOIR ROUTING

**C
B**

CHRISTOPHER B. BURKE ENGINEERING, LTD
10275 WEST HIGGINS ROAD SUITE 460
ROSEMONT, ILLINOIS 60018 (708) 296-0500

CLIENT
TIPPECANOE COUNTY

PROJECT No

APPROVED

TITLE
**FLOW SCHEMATIC OF
EXISTING CONDITIONS**

DATE

EXHIBIT 8.2.2

Chapter 8 - REFERENCES

- 1 Bedient, P. B and Huber, W.C , Hydrology and Floodplain Analysis, Addison-Wesley Publishing Company, 1988
- 2 Hoggan, D. H., Computer-Assisted Floodplain Hydrology and Hydraulics, McGraw-Hill, Inc., 1989
3. Hydrologic Engineering Center, "HEC-1 Flood Hydrograph Package: User's Manual and Programmer's Manual," U.S. Army Corps of Engineers, Davis, California, September 1990.
- 4 McCuen, R H , Hydrologic Analysis and Design, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1989
- 5 Purdue, A M , Jeong, G D , and Rao, A R., "Statistical Characteristics of Short Time Incremental Rainfall," Department of Public Works of Indianapolis/Purdue University, August 1992
- 6 Soil Conservation Service, "TR-20 Computer Program for Project Formulation Hydrology," Technical Release Number 20, May 1982
- 7 Soil Conservation Service, "Urban Hydrology for Small Watersheds," Technical Release 55, 2nd ed , U.S. Dept. Agriculture, NTIS PB87-101580, Springfield, Virginia, June 1986

APPENDIX A

STATISTICAL ANALYSIS

Appendix A - STATISTICAL ANALYSIS

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Appendix A - STATISTICAL ANALYSIS

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Appendix A - STATISTICAL ANALYSIS

LIST OF PARAMETERS

α, u	Functions of the mean and standard deviation in Gumbel's cumulative distribution function
b	Variable in Gumbel's cumulative distribution function
CDF	Cumulative Distribution Function
g	Skewness
J	Risk
k	Gumbel frequency factor
k'	Log Pearson frequency factor as a function of the skewness coefficient and return period
m	Number of occurrences
n	Number of discrete samples
$P(E_1)$	Probability of a single event
$P(E'_1)$	Probability of an event not occurring
s_d	Standard deviation
s'_d	Standard deviation of the logarithmic transformed values
T_r	Return Period
x	Discrete value of a sample or event
X	Magnitude of an event
x_m	Mean of the data
\bar{x}	Mean of the data

Appendix A - STATISTICAL ANALYSIS

This appendix is presented as a summary of the probabilistic and statistical principles used in the analysis of hydrological data. The general concepts are presented and the mode of application outlined. The reader is referred to the references at the end of the chapter for additional information.

A 1 GENERAL CONCEPTS

In the statistical analysis of a process, the process is termed 'continuous' if it can assume any value between the limits of that process or 'discrete' if the process is confined to specified, incremental values. Examples of a continuous process are rainfall, storm runoff and river flows. An example of a discrete process is the number of times a batter in baseball reaches base safely.

In the study of hydrology we are usually concerned with average and extreme values and their variability. This information is utilized in the design of dams, storage facilities, storm sewers, etc.

A 2 - STATISTICAL PARAMETERS

A 2 1 The Mean

The mean of a set is often called the average. It is a measure of the central tendency of a distribution. This is illustrated in Figure A 3 1, where the areas to the right and left of line A-A are equal. A curve such as that shown in Figure A 3.1 is a continuous probability distribution function. For discrete samples, the mean is given by:

$$\bar{X} = x_m = \frac{1}{n} \sum_{i=1}^n x_i \quad (\text{A } 2 \text{ } 1)$$

where x_m is the mean of the x_i data set

The log mean is written as

$$\log x_m = \sum_{i=1}^n \frac{\log x_i}{n} \quad (\text{A.2 } 2)$$

A 2 2 - Standard Deviation

The standard deviation of a sample is indicative of the scatter or the average distance about the mean. These deviations are calculated as $(x_1 - x_m)$, $(x_2 - x_m)$, . . . $(x_n - x_m)$, where x_m is the mean. The average distance is the standard deviation which is written as:

$$S_d = \left[\frac{1}{n-1} \sum_{i=1}^n (x_i - x_m)^2 \right]^{1/2} \quad S'_d = \left[\frac{1}{n-1} \sum (\log x_i - \log x_m)^2 \right]^{1/2} \quad (\text{A.2.3})$$

The skewness indicates the weighting of the distribution to either side of the mean. It is determined by

$$g = \frac{1}{S_d^3} \frac{n}{(n-1)(n-2)} \sum_{i=1}^n [x_i - x_m]^3 \quad (\text{A 2.4})$$

The skewness is zero for symmetrical distributions, greater than zero for right skewness and less than zero for left skewness. The probability distribution in Figure A 3.1 has positive or right skewness.

A 3 - PROBABILITY CONCEPTS

The probability of a single event, E_1 , is defined as the relative number of occurrences of the event after a sufficiently large number of trials. Therefore, the probability of event E_1 , $P(E_1)$, is given as m_1/n for m_1 occurrences of event E_1 in n trials, if n is sufficiently large. The value m_1 represents the frequency of the event and m_1/n is the relative frequency or probability. The following illustrates this concept.

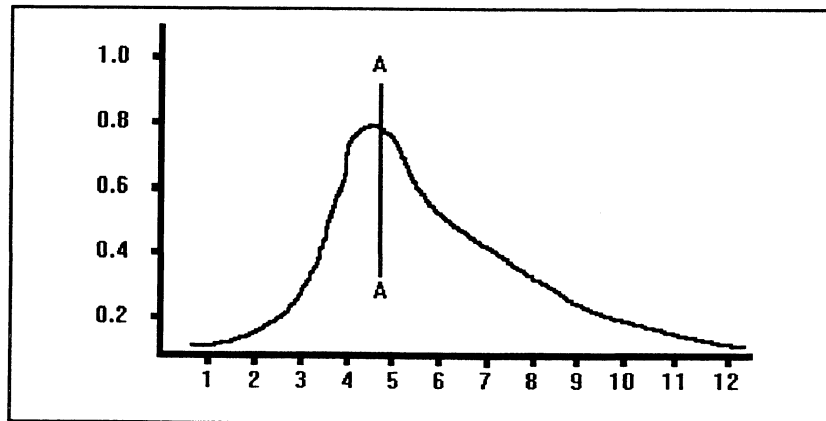


Figure A 3 1
Probability Distribution Function

Table A.3 1 presents the yearly data for the annual peak flow in the Wabash River at Lafayette, Indiana for a period of 69 years. These data represent a continuous process but for convenience it can be discretized in groups of 10,000 cfs intervals

Table A 3 1 Annual Peak Flows for the Wabash River at Lafayette, Indiana - 1924 - 1992

Year	Peak Discharge (cfs)	Year	Peak Discharge (cfs)
1924	59,800	1959	87,600
1925	63,300	1960	38,100
1926	57,700	1961	54,700
1927	64,000	1962	45,000
1928	63,500	1963	60,000
1929	38,000	1964	57,600
1930	74,600	1965	36,000
1931	13,100	1966	64,100
1932	37,600	1967	65,300
1933	67,500	1968	67,100
1934	21,700	1969	64,900
1935	37,000	1970	41,700
1936	93,500	1971	30,000
1937	58,500	1972	38,500
1938	63,300	1973	39,500
1939	74,400	1974	48,800
1940	34,200	1975	34,700
1941	14,600	1976	43,400
1942	44,200	1977	31,400
1943	131,000	1978	49,500
1944	73,300	1979	48,800
1945	46,600	1980	46,200
1946	39,400	1981	43,200
1947	41,200	1982	55,300
1948	41,300	1983	59,100
1949	62,000	1984	40,000
1950	90,000	1985	79,200
1951	50,600	1986	40,000
1952	41,900	1987	13,800
1953	35,000	1988	32,600
1954	16,500	1989	39,500
1955	35,300	1990	50,800
1956	30,000	1991	75,200
1957	52,000	1992	32,700
1958	97,000		

Plotting the occurrences in each 10,000 cfs intervals gives the histogram shown in Figure A 3 2. In the selection of an interval, it is best to choose the smallest one possible. In fact, if the increment is infinitesimal and the record continuous, the bar graph would become a smooth curve similar to Figure A 3 1.

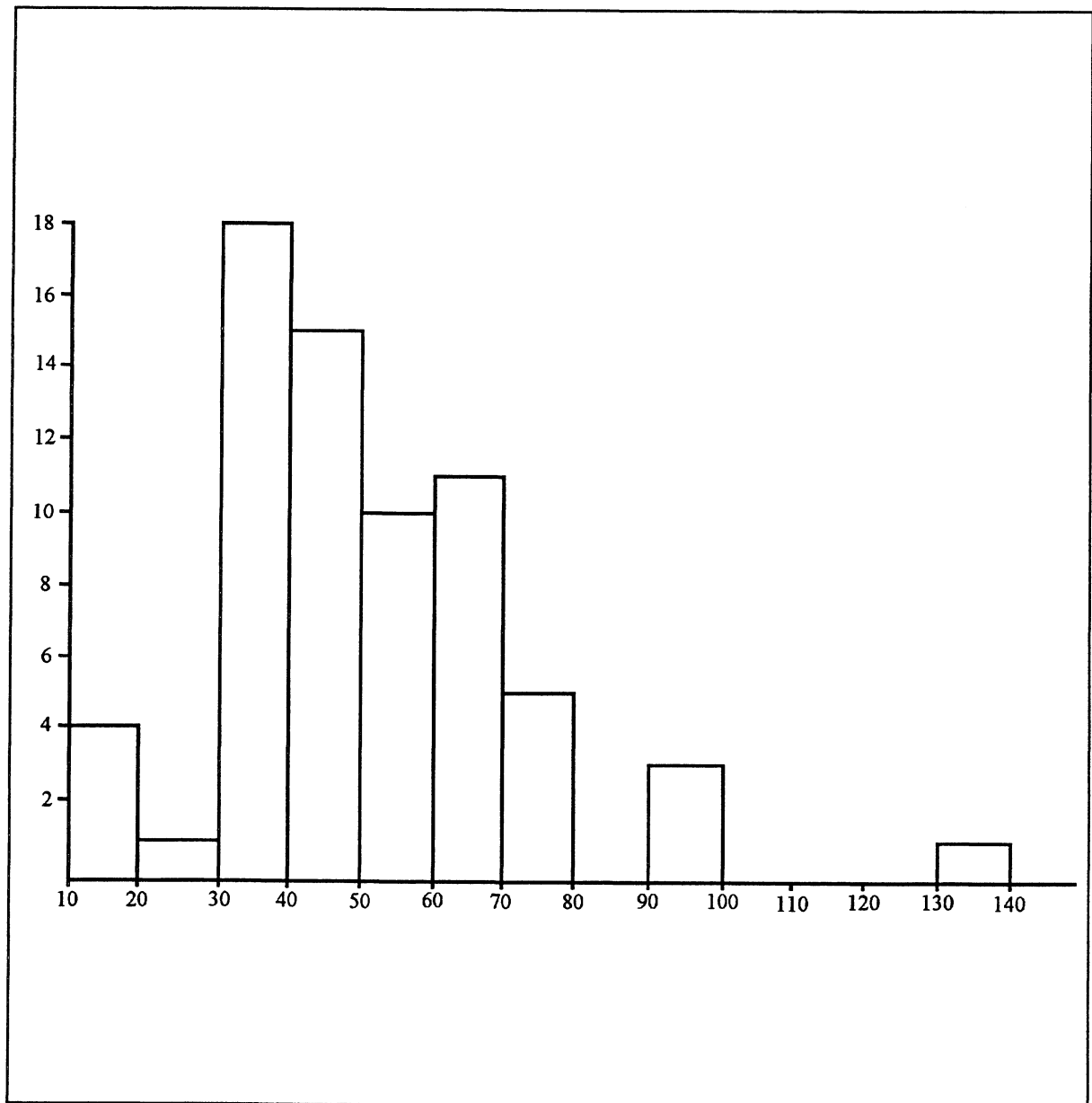


Figure A 3 2
Histogram of the Annual Maximum Flows of the
Wabash River at Lafayette, Indiana (1924 - 1992)

From Figure A.3.2, it can be concluded that on the average, the frequency of an annual peak flow with a magnitude between 30,000 and 40,000 cfs is 18 times in 69 years, where 18 is previously mentioned as m_1

If the number of occurrences of this event is divided by the sample size of 69, the relative frequency of the occurrence of that event class can be determined. The relative frequency for flows with a magnitude between 30,000 and 40,000 cfs is therefore 18/69 or 0.26. But since the number of samples is small, it is only an estimate of the true probability.

If the frequencies of each one of the intervals in Figure A.3.2 are cumulatively summed up and graphed, the curve shown in Figure A.3.3 is found. This curve is a cumulative distribution function (CDF) and shows that the proportion of annual peak flows which were observed to have a magnitude less than or equal to 50,000 cfs, for example, is 0.55.

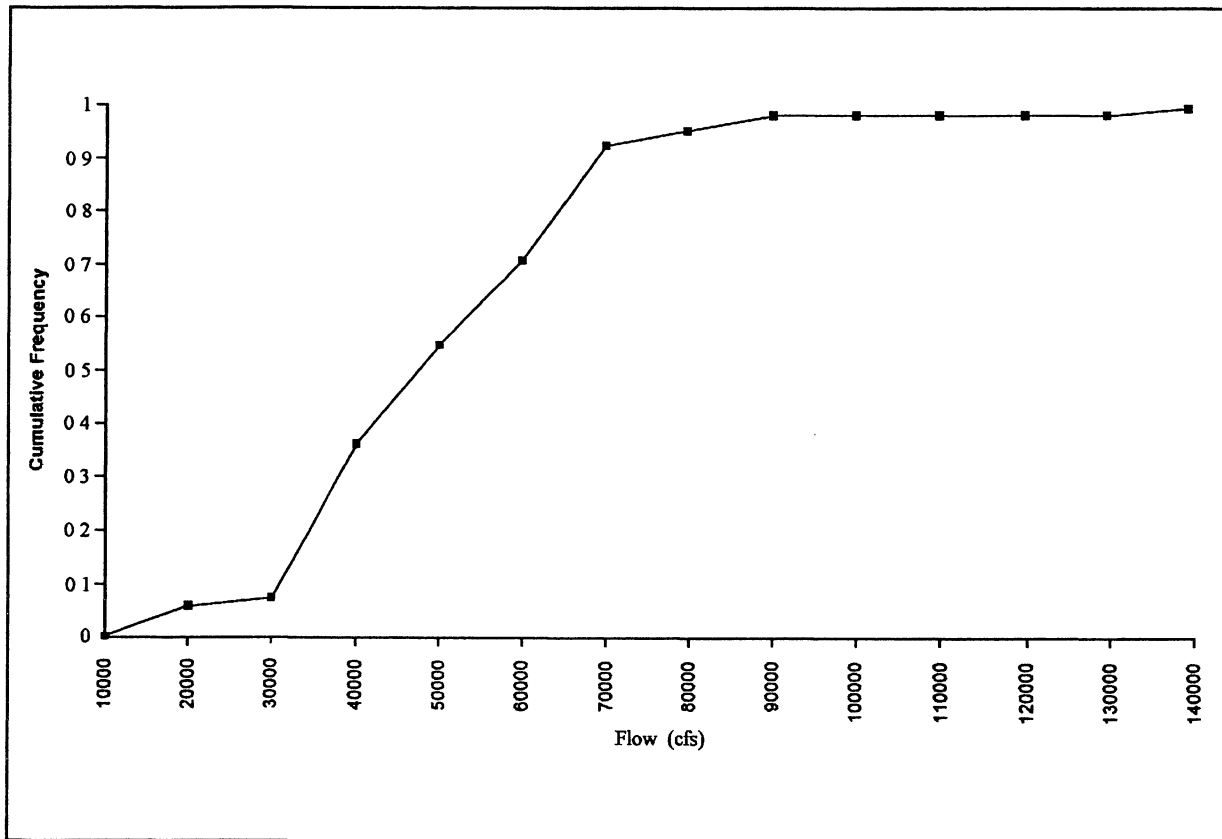


Figure A 3 3
Cumulative Distribution of Maximum Annual Flows of the
Wabash River at Lafayette, Indiana (1924 - 1992)

The probability of an event occurring can be said to equal the relative frequency, and since the probability of an event certain to occur is one and an impossible event has probability zero, we can express the probability of E_1 as falling in the range

$$0 \leq P(E_1) \leq 1 \quad (\text{A } 3.1)$$

It is observed from Figure A.3.3 that the sum of the probability estimates of the events is equal to one.

$$P(E_1) + P(E_2) + P(E_3) + \dots + P(E_n) = \sum_{i=1}^n P(E_i) = 1 \quad (\text{A } 3.2)$$

The probability of an event not occurring $P(E'_1)$ plus the probability of that event occurring $P(E_1)$ is one. This can be written as:

$$P(E'_1) + P(E_1) = 1 \quad (\text{A } 3.3)$$

As an example, take the probability estimate of a flood greater than 50,000 cfs. In this case it would be $1 - 0.55$ or 0.45 .

The information presented in the last two sections provides the manual user with a basic understanding of the statistical and probability techniques used in hydrology.

It should be pointed out that the probability estimate used in this section was given as m/n , where m was the number of occurrences of an event and n was the total number of observations. In later sections the plotting procedures utilize the expression $P = m/(n + 1)$ or $T_r = (n + 1)/m$. The reason for the $(n + 1)$ term rather than n is beyond the scope of this manual. For now, it can be thought of as a correction to compensate for limited sample data so that the population distribution will be more representative.

A 4 - PROBABILITY DISTRIBUTIONS

This section presents two distributions which are used to describe the distribution of hydrological phenomenon. Only the basics of each distribution are presented. If more detail is desired, the user is referred to standard texts on the subject or references at the end of this Appendix.

A 4 1 - Gumbel's Extreme Value Distribution

The Gumbel distribution has been successfully used to represent minimum or maximum values of a given sample. The distribution is asymmetrical like the one shown in Figure A 3 1 with a log-normal distribution and constant skewness of 1.1396. The probability for an event being less than or equal to a given magnitude is given by the cumulative distribution function (CDF) and is written as:

$$P(X \leq x) = F(x) = \exp[-\exp[-\alpha(x-u)]] \quad (\text{A.4 1})$$

where α and u are functions of the mean and standard deviation (X is the given magnitude, x is the event)

The expression $-\alpha(x-u)$ can be estimated by

$$b = -\alpha(x - u) = \frac{(x - x_m + 0.45s_d)}{0.78s_d} \quad (\text{A 4 2})$$

As an example, consider the data shown in Table A.3.1. The mean for this data was 51,000 cfs and the standard deviation is 21,000 cfs. Therefore, for a flood of 50,000 cfs, $b = (50,000 - 51,000 + 0.45(21,000))/(0.78 \times 21,000) = 0.516$. Therefore, $F(X)$ for a 50,000 cfs flood is $\exp[-\exp(-0.516)] = 0.55$. This checks with the observed value of 0.55 shown in Figure A 3 3. The probability of the event being exceeded is $1 - 0.55$ or 0.45. The return period is then given as $1/0.45$ or 2.22 years.

A simplified method for estimating the magnitude of a flood using the Gumbel distribution for a given return period and sample size was presented by Chow, based on Equation A.4 3.

$$x = x_m + ks_d \quad (\text{A 4 3})$$

In Equation A 4 3, k is a frequency factor which may be found by using Equation A 4.4 for a very large sample size distribution, or Table 2.2 1, page 2-19, for a sample size up to 100.

$$k = \frac{\sqrt{6}}{\pi} \left[0.5772 + \ln \left[\ln \frac{T_r}{T_r - 1} \right] \right] \quad (\text{A.4 4})$$

Example 2 2 1 in Chapter 2 demonstrates the use of Equation A 4 3 along with the procedures used in the plotting of the Gumbel distribution.

A 4 2 - Log Pearson Type III Distribution

This distribution is usually the recommended standard method for analyzing hydrologic data (Water Resources Council, 1981). The distribution is skewed and the data is transformed by

$X = \log x$ to reduce the skewness. The mean, standard deviation, and skewness of this transformed data are then used to fit the distribution. The formula used for the distribution is somewhat involved and will not be presented here.

The simplified method used in the analysis is to use the following equation.

$$\log x = \log \bar{x} + k' s'_d \quad (\text{A 4.5})$$

In Equation A.4.5, s'_d corresponds to the standard deviation of the logarithmic transformed values, and term k' is a frequency factor which is a function of the skewness coefficient and return period. Table 2.3 presents these factors as a function of return period. Examples and other discussion of the distribution are found in Chapters 2 and 3. The graphical approach to the analysis is identical to the Gumbel distribution except a different plotting paper is used.

A 5 - RISK

In Section A 3 it was shown that the frequency $P(E)$ represents an average value. Using a flood as an example, we observe a flood magnitude associated with $P(E) = 0.25$ has on average 25 percent chance each year of being equaled or exceeded. It also implies that the flood would occur on the average once in 4 years $[1 / 0.25]$. This is called the return period T_r .

We can determine the probability that the flood will not occur in a year by.

$$P(\bar{E}) = 1 - P(E) = 1 - \frac{1}{T_r} \quad (\text{A.5 1})$$

It follows that the probability that E will not occur for n successive years is:

$$P_1(E_m) \times P_2(E_m) \times \dots \times P_n(E_m) = P(E_m)^n = \left[1 - \frac{1}{T_r}\right]^n \quad (\text{A 5.2})$$

The risk of an event is the probability that the event will occur at least once in n years. As presented by Viessman et al (1989), the risk J is equal to:

$$J = 1 - \left[1 - \frac{1}{T_r}\right]^n = 1 - [P(E_m)]^n \quad (\text{A 5 3})$$

Table A 5 1 presents a tabular solution of Equation A 5.3 for various levels of risk and the expected design life of the structure. For a structure with a design life of 30 years and accepted risk of 10%, the structure should be designed with a return period of 250 years as determined below.

$$0.10 = 1 - \left[1 - \frac{1}{T_r} \right]^{30} \quad 0.9965 - 1 = \frac{-1}{T_r}$$

$$0.90^{\frac{1}{30}} = 1 - \frac{1}{T_r} \quad T_r = 285 \text{ years}$$

Table A.5.1
Return Period Associated with Various Degrees of Risk and
Expected Design Life (after Viessman et al., 1989)

Risk %	Expected Design Life (years)							
	2	5	10	15	20	25	50	100
75	2 00	4 02	6 69	11 0	14.9	18 0	35.6	72.7
50	3 43	7 74	14.9	22 1	29 4	36.6	72.6	144.72
40	4 44	10 3	20.1	29 9	39.7	49 5	98.4	196.3
30	6 12	14 5	28.5	42 6	56 5	70.6	140.7	281
25	7.46	17.9	35 3	52.6	70.0	87 4	174 3	348
20	9 47	22 9	45.3	67.7	90.1	112.5	224.6	449
15	12 8	31 3	62 0	90 8	123 6	154.3	308	616
10	19 5	48 1	95 4	142.9	190 3	238	475	950
5	39 5	98 0	195 5	292 9	390	488	976	1949
2	99 5	248	496	743	990	1238	2475	4950
1	198 4	498	996	1492	1992	2488	4975	9953

The following pages contain rainfall depth maps for the entire continental United States. They are referenced in Chapter 2, Section 2.2.5 for determining rainfall depths for locations where Intensity-Duration-Frequency curves have not been developed.

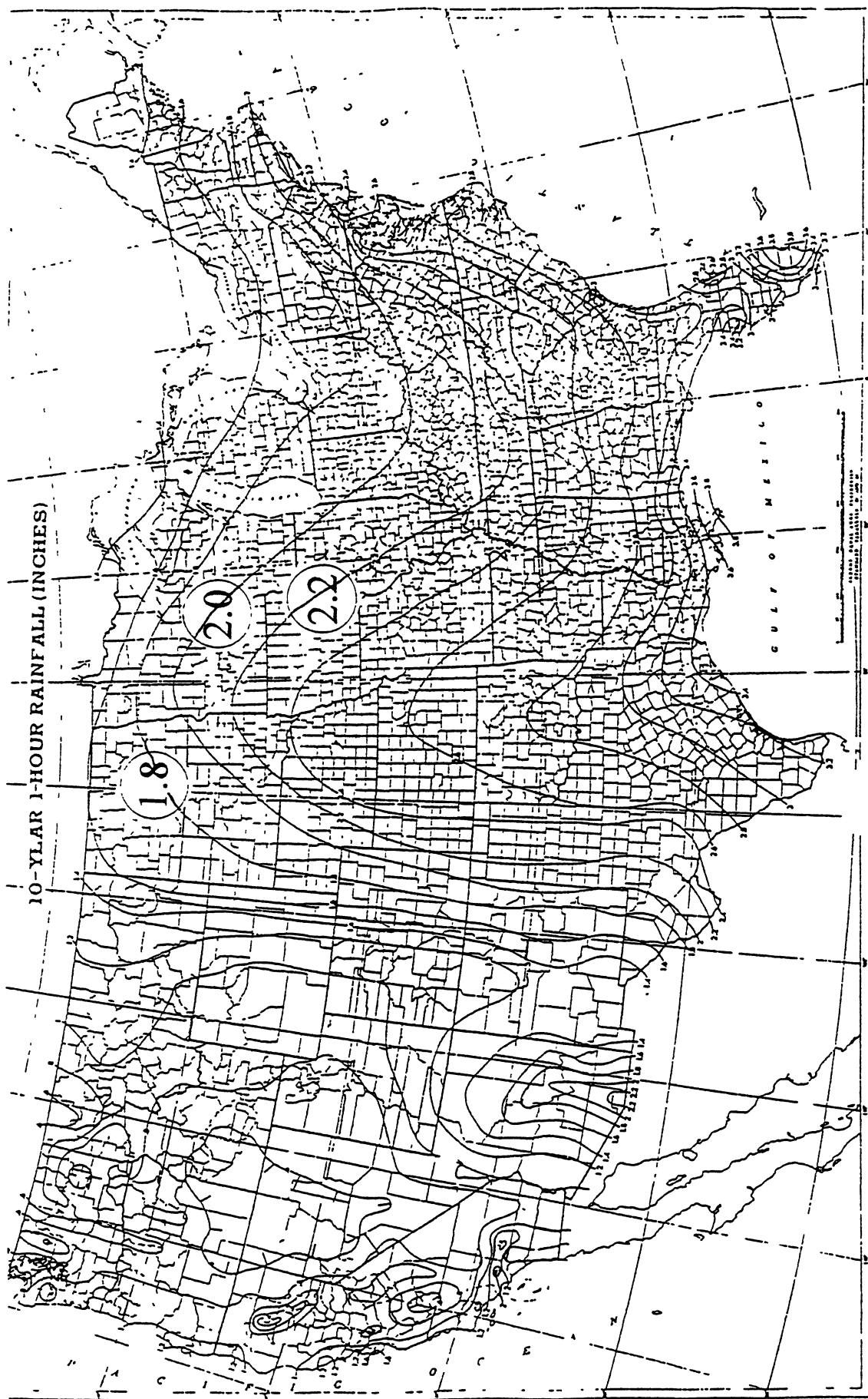


Figure A.6.1
10-Year 1-Hour Rainfall Depth Map (U.S. Weather Bureau Technical Paper 40)

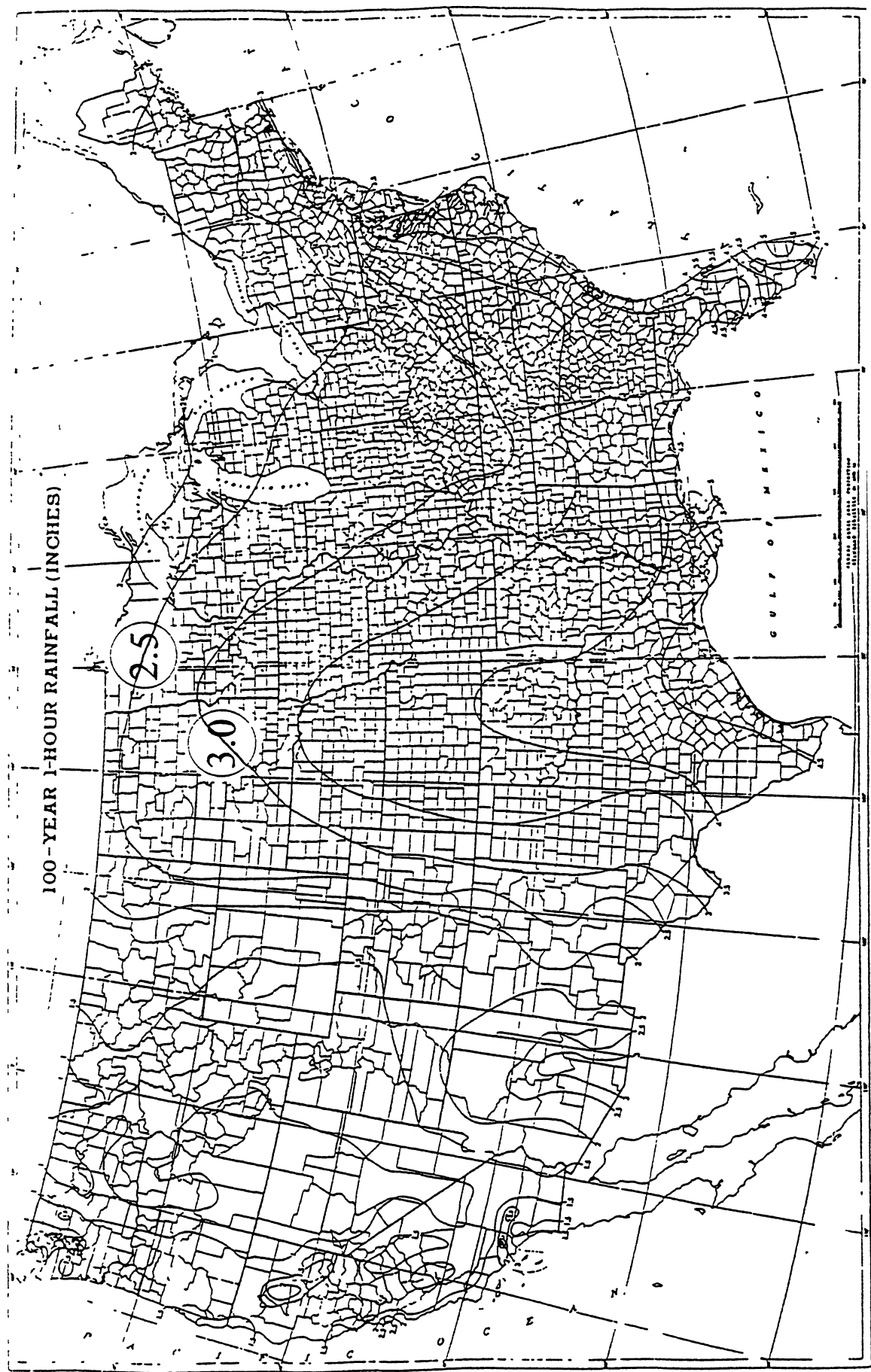


Figure A.6.3
100-Year 1-Hour Rainfall Depth Map (U.S. Weather Bureau Technical Paper 40)

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APPENDIX B

FUNDAMENTALS OF HYDRAULICS

Appendix B - FUNDAMENTALS OF HYDRAULICS

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Appendix B - FUNDAMENTALS OF HYDRAULICS

LIST OF PARAMETERS

A	Area (ft ²)
C	Coefficient for Hazen-Williams equation
C _d	Coefficient of discharge
D	Diameter of pipe (ft)
D'	Hydraulic depth (ft)
EGL	Energy grade line
ε	Equivalent roughness (ft)
f	Friction factor
g	Acceleration due to gravity (32.2 ft/sec ²)
γ	Specific weight of the fluid (lbs/ft ³)
h	Vertical distance from the fluid surface (ft)
h _f	Pipe friction
h _L	Head loss due to pipe friction (ft)
h _m	Minor head losses (ft)
h _w	Height of water above weir crest (ft)
H _A	Mechanical energy head added to the system
H _R	Mechanical energy head removed from the system
HGL	Hydraulic grade line
L	Length of pipe (ft)
L _e	Equivalent length of pipe (ft)
m	Mass flowrate (slugs/sec)
n	Manning's roughness coefficient
ν	Kinematic viscosity of the fluid (ft ² /sec)
P	Hydrostatic pressure (lbs/ft ²)
P'	Wetted perimeter (ft)
Q	Flowrate (cfs)
R	Hydraulic radius (ft)
R _e	Reynolds number
ρ	Fluid density (slugs/ft ³)
S	Slope (ft/ft)
T	Top-width (ft)
V	Velocity (ft/sec)

Appendix B - FUNDAMENTALS OF HYDRAULICS

This appendix presents the fundamental principles of hydraulics and the basic concepts behind flow in pipes and open channels. These augment the materials presented elsewhere in the manual.

B.1 - CONSERVATION OF MASS

For steady flow (flow which does not vary with time), the principle of conservation of mass states that the flow of mass into a control volume must equal the flow of mass out of control volume for a given increment of time. This may be written between any two points within the control volume as:

$$m = \rho_1 A_1 V_1 = \rho_2 A_2 V_2 \quad (\text{B.1.1})$$

where subscripts 1 and 2 denote two sections in the flow, m = mass flow rate (slugs/sec), ρ = fluid density (slugs/ft³), V = average velocity across the section (ft/sec), and A = area normal to the section (ft²).

For fluids such as water which are incompressible, the density remains a constant ($\rho_1 = \rho_2$). Thus, the continuity equation is obtained.

$$Q = A_1 V_1 = A_2 V_2 \quad (\text{B.1.2})$$

where Q = volumetric flowrate (ft³/s) (cfs).

B.2 - PRESSURE

The hydrostatic (stationary fluid) pressure varies directly with the fluid depth:

$$P = \gamma h \quad (\text{B.2.1})$$

where P = hydrostatic pressure (lbs/ft²), γ = specific weight of the fluid (lbs/ft³), and h = vertical distance from the fluid surface (ft).

This relationship is shown graphically in Figure B.2.1.

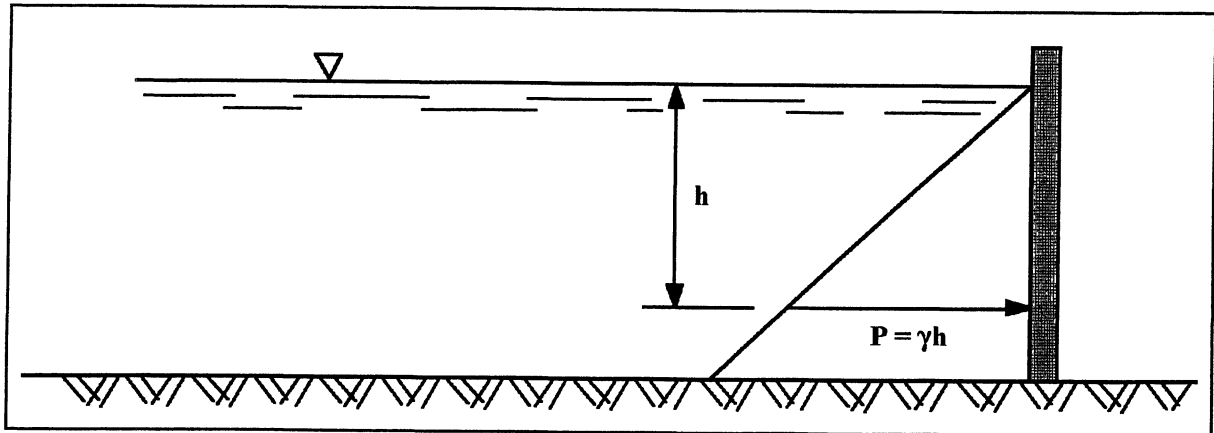


Figure B.2.1
Graphical Representation of Pressure Variation with Depth

The pressure determined by Equation B.2.1. and shown in Figure B.2.1, is termed gage pressure. Gage pressure is pressure taken from a reference scale with atmospheric pressure as zero. When the pressure is measured with respect to a perfect vacuum, it is called absolute pressure. Figure B.2.2 shows the difference between the two pressure reference planes when the pressure is expressed in pounds per square inch (psi). Gage pressure is denoted as psig and absolute as psia.

Gage Pressure		Absolute Pressure
20 psig		34.7 psig
0 psig	Normal Atmospheric Pressure	14.7 psig
-14.7 psig	Perfect Vacuum	0 psig

Figure B.2.2
Gage and Absolute Pressure

B.3 - ENERGY

The three types of energy which moving fluids may possess are: kinetic, potential and pressure. These are represented by the following relationships.

- Kinetic Energy/weight = $\frac{V^2}{2g}$ (ft-lbs/lbs) or (ft)

where g = acceleration due to gravity (32.2 ft/sec²)

- Potential Energy/weight = z (ft-lbs/lb) or (ft)
where z = elevation above some arbitrary datum
- Pressure Energy/weight = $\frac{P}{\gamma}$ (ft-lbs/lb) or (ft)

The energy of a fluid flowing across two sections (1 and 2) in a fluid continuum can be equated by writing the energy equation.

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + z_1 + H_A + H_R - h_L = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + z_2 \quad (\text{B.3.1})$$

where H_A = mechanical energy head (ft) added to the fluid system between points 1 and 2 (i.e. pump), H_R = mechanical energy head (ft) removed from the fluid system between points 1 and 2 (i.e. turbine), and h_L = head loss (ft) due to pipe friction (h_f) and obstructions such as valves, bends, etc. (minor losses, h_m) ($h_L = h_f + h_m$) between points 1 and 2.

The energy grade line (EGL) can be drawn graphically to represent the energy loss or gain along a pipe or channel. This is shown in Figure B.3.1 along with the hydraulic grade line (HGL), which is the sum of the pressure head and potential energy head [$P/\gamma + z$].

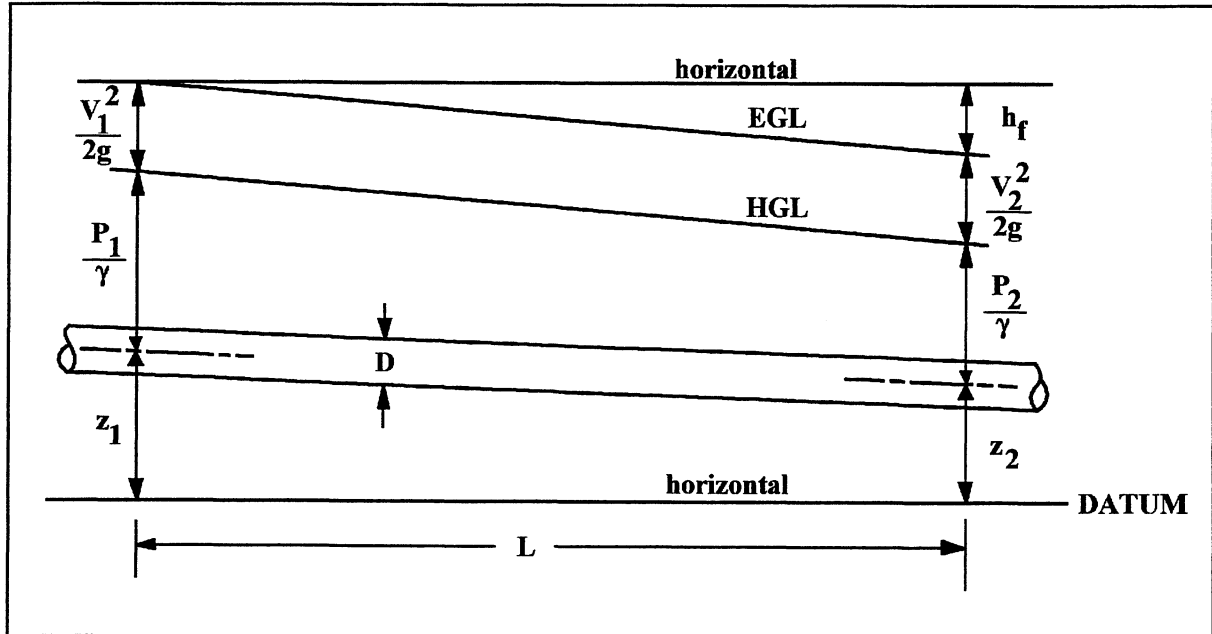


Figure B.3.1
Graphical Representation of Energy Grade Line and Hydraulic Grade Line for Pipe Flow

B 4 - FLOW IN PIPES UNDER PRESSURE

Pipe flow problems can usually be solved using the continuity and energy equation along with some method to evaluate the h_f term. There are basically two methods by which h_f may be determined: the Darcy-Weisbach and Hazen-Williams equations.

B.4.1 - Darcy-Weisbach Equation

This equation is very general and may be applied to any fluid. It is written as:

$$h_f = f \frac{L}{D} \frac{V^2}{2g} \quad (\text{B.4.1})$$

where L = length of pipe (ft), D = pipe diameter (ft), and f = friction factor.

The friction factor, f , may be found using the Moody Diagram (Figure B.4.1) by calculating the Reynolds number, R_e , and the relative roughness, ϵ/D , where ϵ = the equivalent roughness of the pipe wall, (ft), given in Table B.4.1, D = pipe diameter (ft), $R_e = VD/\nu$, and ν = kinematic viscosity of the fluid (ft^2/sec) (values for water are given in Table B.4.2).

The friction factor is determined by entering the graph along with the ϵ/D curve and moving left along the curve until the curve intersects a vertical line for the given R_e (from bottom axis). Read the left vertical axis to determine the friction factor.

The Darcy-Weisbach Equation can be rewritten to find the diameter and flowrate as presented in Equations B.4.2 and B.4.3.

$$D = \left[\frac{8 L Q^2 f}{\pi^2 g h_f} \right]^{1/5} \quad (\text{B.4.2})$$

$$Q = \left[\frac{\pi^2 D^5 g h_f}{8 L f} \right]^{1/2} \quad (\text{B.4.3})$$

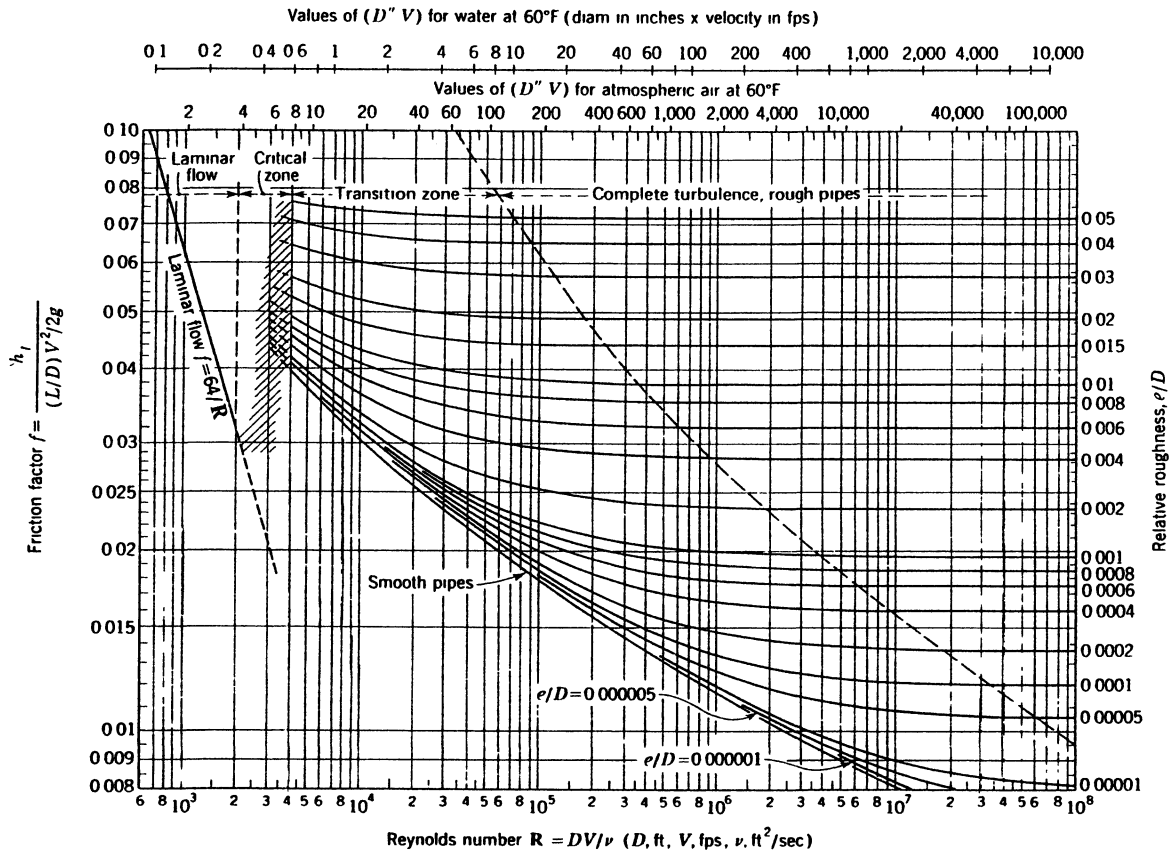


Figure B.4.1
Moody Diagram (Daugherty et al., 1985)

Another approach in estimating f is to use the explicit formula developed by Swamee and Jain (1976).

$$f = \frac{1.325}{\left[\ln \left(\frac{\epsilon}{3.7D} + \frac{5.74}{R_e^{0.9}} \right) \right]^2} \quad (\text{B.4.4})$$

$$\text{valid for} \quad 10^{-6} \leq \frac{\epsilon}{D} \leq 10^{-2} \quad 5000 \leq R_e \leq 10^8$$

Table B.4.1
Equivalent Roughness

Pipe Material	ϵ (feet)
Riveted steel, few rivets	0.003
Riveted steel, many rivets	0.030
Concrete, finished surface	0.001
Concrete, rough surface	0.010
Wood-stave, smooth surface	0.0006
Wood-stave, rough surface	0.003
Cast iron, new	0.00085
Galvanized iron, new	0.00050
Asphalted cast iron, new	0.00040
Commercial steel, new	0.00015
Wrought iron, new	0.00015
Drawn tubing, new	0.000005
(glass, brass, copper, lead)	(essentially "smooth")

Table B.4.2
Values of Kinematic Viscosity for Water

Temp. °F	Kinematic Viscosity (ft ² /sec)
40	1.67×10^{-5}
50	1.40×10^{-5}
60	1.21×10^{-5}
70	1.05×10^{-5}
80	9.15×10^{-6}
90	8.39×10^{-6}
100	7.36×10^{-6}

B.4.2 - Hazen-Williams Equation

This equation is only applicable for water at normal temperatures and for pipes with a diameter greater than 2 inches and a velocity less than 10 feet per second. The various forms of the equation are given below:

$$h_f = 10.6 (Q_{\text{mgd}}/C)^{1.85} L/(D)^{4.87} \quad (\text{B.4.5})$$

$$h_f = 4.73 (Q_{\text{cfs}}/C)^{1.85} L/(D)^{4.87} \quad (\text{B.4.6})$$

$$Q_{\text{cfs}} = 0.432 C D^{2.63} (h_f/L)^{0.54} \quad (\text{B.4.7})$$

$$Q_{\text{mgd}} = 0.279 C D^{2.63} (h_f/L)^{0.54} \quad (\text{B.4.8})$$

where subscripts cfs and mgd denote flow in cubic feet per second and million gallons per day, respectively.

C = coefficient which is dependent upon surface roughness (see Table B.4.3).

B.4.3 - Minor Losses

Minor losses are losses due to fittings, obstructions, expansions or contractions. The most convenient method of quantifying these losses is the equivalent length technique. This technique expresses the minor loss in terms of an equivalent length of pipe which would have the same loss. This is expressed as:

$$h_m = f \frac{L_e}{D} \frac{V^2}{2g}$$

where L_e is the equivalent length of pipe.

Table B.4.4 contains a list of equivalent lengths expressed as L_e/D . Knowing the diameter of the minor loss, the equivalent corresponding length can be found.

B.4.4 - Flow in a Series Pipe System

For a pipe system which is in series, the discharge in each pipe is identical (assuming no inflow or outflow). Therefore, the total energy loss in the system is the sum of the head loss in each pipe.

$$h_f = h_{f1} + h_{f2} + \dots + h_{fn}$$

This type of problem is solved by calculating the flow and headloss in each pipe and summing all the losses.

Table B.4.3
Values of C for the Hazen-Williams Equation

Type of Pipe	Condition	C
Cast Iron	New	All sizes 130
	5 years old	12" and Over 120
		8" 119
		4" 118
	10 years old	24" and Over 113
		12" 111
		4" 107
	20 years old	24" and Over 100
		12" 96
		4" 89
	30 years old	30" and Over 90
		16" 87
		4" 75
	40 years old	30" and Over 83
		16" 80
		4" 64
	50 years old	40" and Over 77
		24" 74
		4" 55
Welded Steel	Values of C the same as for cast iron pipes, 5 years older	
Riveted Steel	Values of C the same as for cast iron pipes, 10 years older	
Wood Stave	Average value, regardless of age	120
Concrete or Concrete-lined	Large sizes, good workmanship, steel forms	140
	Large sizes, good workmanship, wooden forms	120
	Centrifugally spun	135
Vitrified	In good condition	110
New High-Density Polyethylene Pipe		155
New PVC pipe		150
Transite (6")		140

Table B.4.4
Representative Equivalent Length in Pipe Diameters (L/D) of Various Valve Fittings
(Flow of Fluids Through Valves, Fittings and Pipe,
Technical Paper 410, The Crane Company, 1969)

Description of Product				Equivalent Length In Pipe Diameters (L/D)			
Globe Valves	Stem Perpendicular to Run	With no obstruction in flat, bevel, or plug type seat	Fully open	340			
		With wing or pin guided disc	Fully open	450			
	Y-Pattern	(No obstruction in flat, bevel, or plug type seat)					
		- With stem 60 degrees from run of pipe line - With stem 45 degrees from run of pipe line	Fully open Fully open	175 145			
Angle Valves		With no obstruction in flat, bevel, or plug type seat With wing or pin guided disc	Fully open Fully open	145 200			
Gate Valves	Wedge, Disc, Double Disc, or Plug Disc		Fully open Three-quarters open One-half open One-quarter open	13 35 160 900			
		Pulp Stock		Fully open Three-quarters open One-half open One-quarter open	17 50 260 1200		
			Conduit Pipe Line Gate, Ball, and Plug Valves			Fully open	3
			Check Valves	Conventional Swing	0.5† ... Fully open	135	
	Clearway Swing			0.5† ... Fully open	50		
	Globe Lift or Stop; Stem Perpendicular to Run or Y-Pattern	2.0† ... Fully open		Same as Globe			
	Angle Lift or Stop	2.0† ... Fully open		Same as Angle			
	In-Line Ball	2.5 vertical and 0.25 horizontal† ... Fully open		150			
Foot Valves with Strainer		With poppet lift-type disc With leather-hinged disc	0.3† Fully open 0.4† Fully open	420 75			
Butterfly Valves (8-inch and larger)			Fully open	40			
Cocks	Straight-Through	Rectangular plug port area equal to 100% of pipe area	Fully open	18			
	Three-Way	Rectangular plug port area equal to 80% of pipe area (fully open)	Flow straight through Flow through branch	44 140			
Fittings	90 Degree Standard Elbow			30			
	45 Degree Standard Elbow			16			
	90 Degree Long Radius Elbow			20			
	90 Degree Street Elbow			50			
	45 Degree Street Elbow			26			
	Square Corner Elbow			57			
	Standard Tee	With flow through run With flow through branch		20 60			
		Close Pattern Return Bend			50		

B.4.5 - Flow in a Parallel Pipe System

For a parallel pipe system, the headloss in each pipe is the same and the total flow is equal to the sum of the flow through the individual pipes.

$$\begin{aligned} Q_{\text{TOTAL}} &= Q_1 + Q_2 + Q_3 + \dots + Q_n \\ h_{f1} &= h_{f2} = h_{f3} = h_{fn} \end{aligned}$$

This type of problem is solved by knowing the total flowrate and assuming a distribution of flow in the various pipes. Then the headloss is calculated for each pipe. If the losses are not equal, the flows are adjusted until they converge.

B.5 - FLOW IN OPEN CHANNELS

An open channel is a conveyance system in which the liquid stream is not completely enclosed by solid boundaries, thus the stream has a free surface subjected only to atmospheric pressure. Since the pressure is zero (gage pressure) at the surface, the energy available to cause fluid flow is due to change in elevation from one section to another in the channel. Therefore, the driving force for the flow is that component of the liquid weight which is along the slope of the channel. This driving force is resisted by a shearing force transmitted from the boundaries to the liquid.

B.5.1 - Geometric Properties of Open Channels

In order to properly evaluate the flow in open channels, certain geometric properties need to be defined. These properties which are defined below are given in Table 4.1.1 for various types of channels.

- A = the cross-sectional water area (ft²)
- P' = the wetted perimeter (ft) (the length of the cross-section boundary which is in contact with the fluid)
- R = the hydraulic radius (ft) ($R = A/P'$)
- T = the top-width (ft) (the distance across the free surface of the cross section)
- D' = the hydraulic depth (ft) ($D' = A/T$)

B.5.2 - Flow Calculations for Open Channels

For steady, uniform flow (flow in which the flowrate and water depth do not change from one section to another) of water the Manning Equation is used to calculate the flowrate or velocity. It may be written as

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \quad (\text{B.5.1})$$

where n = roughness coefficient (given in Table 4.2.1).

Equation B 5.1 can be manipulated so that any of the unknown values can be found using known values.

Sometimes it is useful to separate the elements of Equation B.5.1 which depend upon the channel geometry from those which are normally known. This is accomplished by using the section factor $AR^{2/3}$

$$AR^{2/3} = \frac{Q * n}{1.49 S^{1/2}} \quad (\text{B 5.2})$$

Therefore by using a trial and error procedure, the required channel dimensions can be determined

B.5 3 - Weir Flow

A sharp-crested weir is an obstruction placed in an open channel so that the fluid backs up and falls through a notch in the weir face. There are many types of weirs that can be used. The general relationship for a rectangular weir is given by:

$$Q = \frac{2}{3} C_d \sqrt{2g} L h_w^{3/2} \quad (\text{B.5.3})$$

where C_d = discharge coefficient, L = width of weir crest (ft), and h_w = depth upstream of weir (ft). (C_d , which depends on the type and shape of the weir, may be found in various books on fluid mechanics or from weir manufacturers or by direct calibration.)

Refer to Chapter 6, Section 6.3.6 for additional information on shapes of weir openings.

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APPENDIX C

REGULATORY AGENCIES FOR DRAINAGE PROJECTS

Appendix C - REGULATORY AGENCIES FOR DRAINAGE PROJECTS

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Appendix C - REGULATORY AGENCIES FOR DRAINAGE PROJECTS

The purpose of this appendix is to provide the designer with a general list of regulatory agencies, Federal, State and local, which are commonly involved in drainage projects. This list is not necessarily exhaustive, but is presented to give a general overview of the primary regulatory agencies that may have jurisdiction over a proposed drainage project.

C.1 - LOCAL REGULATORY AGENCIES

The designer should specifically check with each Local Regulatory Agency to determine if there are any local drainage ordinances that will affect the design requirement of the proposed storm drainage project.

County Drainage Board: Each county in Indiana has a County Drainage Board which has the primary responsibility for the construction, reconstruction and maintenance of all legal drains, with the exception of areas where the Board has relinquished the responsibility to sanitary districts, conservancy districts, towns or cities or any legal entity responsible for flood control and drainage (IC 19-4-1-1 thru 19-4-1-5).

County Surveyor: Each county in Indiana has a County Surveyor who has the responsibility of investigating, evaluating and surveying all legal drains. The County Surveyor also has the duty of preparing reports, plans, and profiles required for proposed land drainage improvements. In addition the County Surveyor is responsible for the removal of any obstructions from legal drains and repairing any damage, and permitting private drains to be connected to the legal drain (IC 19-4-1-9, 19-4-1-10).

City or Town Engineer: Cities or towns may employ an engineer or contract with a consulting firm to review and approve all proposed plans and documents for drainage projects. For large cities this responsibility may also be shared in part with the Board of Public Works.

Conservancy District: A conservancy district may be established for flood control or prevention, or improving drainage (IC 13-3-3-2). The district will have the authority to approve all plans and documents for a proposed drainage project.

A listing of all County officials may be obtained by calling the HERPICC office at (800) 428-7639.

C.2 - STATE REGULATORY AGENCIES

Whenever a drainage project encompasses or crosses land owned by the state, the department which has jurisdiction over that land has to be notified for approval; i.e., State Highway Department, State Park Service, etc. The following is a list of some state agencies that have some direct control over drainage projects.

Department of Natural Resources: The state of Indiana has created the Natural Resources Commission (NRC) to develop a policy of protecting water resources and preventing or limiting of floods and flood damage. The commission is authorized and empowered to represent and act for and in behalf of the state of Indiana, subject to the approval of the governor, in all matters of flood control and water resources of the state, with any state or Federal agency; to cooperate with, obtain, approve and/or accept any flood control works from and through the Corps of Engineers of the United States Army; and to cooperate with and obtain, approve and/or accept any works or grant of any character or description from and through any agency of the United States relating to flood control and water resources and to administer the funds in connection therewith (IC 13-2-22-12).

Permission must be obtained from the NRC anytime a structure, deposit or obstruction is placed in a floodway (IC 13-2-22-13).

Indiana Department of Environmental Management: The Department has been created and charged to preserve, protect and enhance the quality of the state's environment and to develop and operate programs for the most beneficial use of the resources of the state. The Department is to evolve standards and develop regulations to preserve, protect and enhance the quality of the environment. All parties discharging or proposing to discharge or emit contaminants affecting quality must furnish technical reports and receive approval as directed by the Department (IC 13-7-1-1).

Stream Pollution Control Board: If the connection of a private drain could cause or add to pollution of the receiving stream, written approval must be obtained from the Stream Pollution control board and filed with the County Drainage Board (IC 19-4-6-7).

C.3 - FEDERAL AGENCIES

While there are many Federal agencies which may have input to proposed drainage projects, the primary ones for Indiana are the Army Corps of Engineers and the Soil Conservation Service.

Corps of Engineers: The Corps will participate in urban flood-damage-reduction projects if certain criteria are met. Essentially, the drainage area must be greater than 1.5 square miles, or the flood discharge for a ten-year storm must be greater than 800 cfs. There are certain exceptions to these guidelines which are outlined in Regulation No. 1165-2-21. If these criteria are not met, all improvements are considered part of the storm sewer system and are the responsibility of local authorities (Regulation ER 1165-2-21).

There are three district offices which have jurisdiction in Indiana depending on location. The addresses for these are given below:

Chicago District
111 N. Canal St.
Chicago, IL 60606
Phone: (312) 353-6400

Louisville District
P.O. Box 59
Attn: CEORLOR-F
Louisville, KY 40201
Phone: (502) 582-5607

Detroit District
P.O. Box 1027
Detroit, MI 48231
Phone: (313) 226-6813

There are many Soil Conservation Service offices throughout the State of Indiana. The state has been divided into 5 areas with an Area office in each division. The following list contains the addresses for the State office in Indianapolis and the 5 Area offices. There are also field offices in each county which can be found by calling one of the Area offices (SCS, July 1993).

State Office: U.S. Department of Agriculture
Soil Conservation Service
6013 Lakeside Boulevard
Indianapolis, Indiana 46278-2933
Phone (317) 290-3200
Fax (317) 290-3225

Area Offices:

Area 1: 800 S. College Ave., Suite 1
Rensselaer, Indiana 47978-3009
Phone (219) 866-5840
Fax (219) 866-8916

Area 2: 2060 Riverfork Drive, W
Huntington, Indiana 46750-9004
Phone (219) 356-9560
Fax (219) 356-9557

Area 3: Route 3, Box 50
Greencastle, Indiana 46135-1457
Phone (317) 653-9783
Fax (317) 653-9455

Area 4: 680 Rumbach Avenue
Jasper, Indiana 47546-3513
Phone (812) 634-2103
Fax (812) 634-2104

Area 5: 2600 N. State Highway 7
North Vernon, Indiana 47265-9109
Phone (812) 346-4236
Fax (812) 346-4237

APPENDIX D

TR-20 AND TR-55 BLANK FORMS

Chapter 3: Time of concentration and travel time

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c , thereby increasing the peak discharge. But T_c can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading.

Factors affecting time of concentration and travel time

Surface roughness

One of the most significant effects of urban development on flow velocity is less retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by urban development: the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

Channel shape and flow patterns

In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

Slope

Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water

management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

Computation of travel time and time of concentration

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time (T_t) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600 V} \quad [\text{Eq. 3-1}]$$

where

T_t = travel time (hr),

L = flow length (ft),

V = average velocity (ft/s), and

3600 = conversion factor from seconds to hours.

Time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t1} + T_{t2} + \dots T_{tm} \quad [\text{Eq. 3-2}]$$

where

T_c = time of concentration (hr) and

m = number of flow segments.

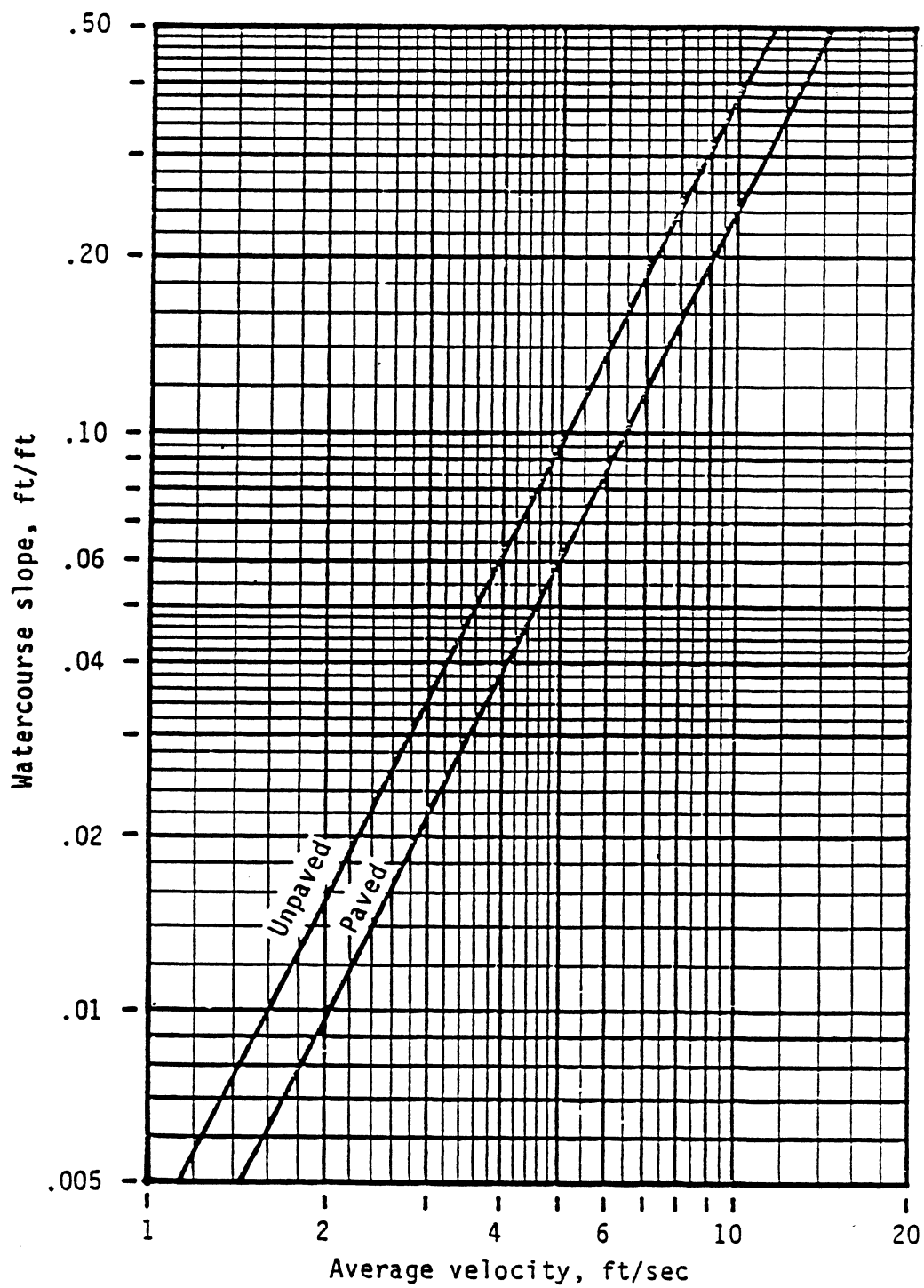


Figure 3-1.—Average velocities for estimating travel time for shallow concentrated flow.

Sheet flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 3-1 gives Manning's n values for sheet flow for various surface conditions.

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overton and Meadows 1976) to compute T_t :

$$T_t = \frac{0.007 (nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \quad [\text{Eq. 3-3}]$$

Table 3-1.—Roughness coefficients (Manning's n) for sheet flow

Surface description	n^1
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover $\leq 20\%$	0.06
Residue cover $> 20\%$	0.17
Grass:	
Short grass prairie	0.15
Dense grasses ²	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods: ³	
Light underbrush	0.40
Dense underbrush	0.80

¹The n values are a composite of information compiled by Engman (1986).

²Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

³When selecting n , consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

where

T_t = travel time (hr),

n = Manning's roughness coefficient (table 3-1),

L = flow length (ft),

P_2 = 2-year, 24-hour rainfall (in), and

s = slope of hydraulic grade line (land slope, ft/ft).

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

Shallow concentrated flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

Manning's equation is

$$V = \frac{1.49 r^{2/3} s^{1/2}}{n} \quad [\text{Eq. 3-4}]$$

where

- V = average velocity (ft/s),
- r = hydraulic radius (ft) and is equal to a/p_w ,
- a = cross sectional flow area (ft²),
- p_w = wetted perimeter (ft),
- s = slope of the hydraulic grade line (channel slope, ft/ft), and
- n = Manning's roughness coefficient for open channel flow.

Manning's n values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using equation 3-4, T_t for the channel segment can be estimated using equation 3-1.

Reservoirs or lakes

Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero.

Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation 3-3 was developed for use with the four standard rainfall intensity-duration relationships.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c . Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.
- The minimum T_c used in TR-55 is 0.1 hour.

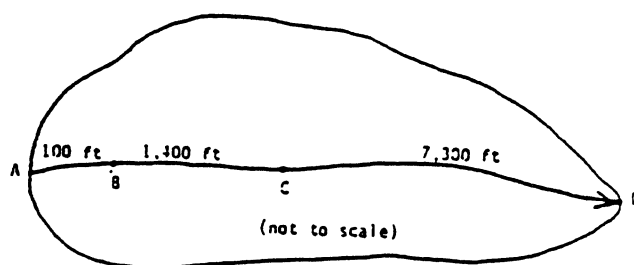
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. The procedures in TR-55 can be used to determine the peak flow upstream of the culvert. Detailed storage routing procedures should be used to determine the outflow through the culvert.

Example 3-1

The sketch below shows a watershed in Dyer County, northwestern Tennessee. The problem is to compute T_c at the outlet of the watershed (point D). The 2-year 24-hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute T_c , first determine T_t for each segment from the following information:

- Segment AB: Sheet flow; dense grass; slope (s) = 0.01 ft/ft; and length (L) = 100 ft.
- Segment BC: Shallow concentrated flow; unpaved; s = 0.01 ft/ft; and L = 1400 ft.
- Segment CD: Channel flow; Manning's n = .05; flow area (a) = 27 ft²; wetted perimeter (p_w) = 28.2 ft; s = 0.005 ft/ft; and L = 7300 ft.

See figure 3-2 for the computations made on worksheet 3.



Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed

Circle one: T_c T_c through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

Segment ID

1. Surface description (table 3-1)
2. Manning's roughness coeff., n (table 3-1) ..
3. Flow length, L (total L \leq 300 ft) ft
4. Two-yr 24-hr rainfall, P_2 in
5. Land slope, s ft/ft
6. $T_t = \frac{0.007 (nL)^{0.8}}{P_2^{0.5} s^{0.4}}$ Compute T_t hr

Shallow concentrated flow

Segment ID

7. Surface description (paved or unpaved)
8. Flow length, L ft
9. Watercourse slope, s ft/ft
10. Average velocity, V (figure 3-1) ft/s
11. $T_t = \frac{L}{3600 V}$ hr

$+$

$=$

Channel flow

Segment ID

12. Cross sectional flow area, a ft^2
13. Wetted perimeter, p_w ft
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ft
15. Channel slope, s ft/ft
16. Manning's roughness coeff., n
17. $V = \frac{1.49 r^{2/3} s^{1/2}}{n}$ Compute V ft/s
18. Flow length, L ft
19. $T_c = \frac{L}{3600 V}$ Compute T_c hr
20. Watershed or subarea T_c or T_c (add T_c in steps 6,

	+		=	
--	---	--	---	--

1, and 19) hr

Worksheet 2: Runoff curve number and runoff

Project _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed _____

1. Runoff curve number (CN)

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN ^{1/}			Area <input type="checkbox"/> acres <input type="checkbox"/> mi ² <input type="checkbox"/> %	Product of CN x area
		Table 2-2	Fig. 2-3	Fig. 2-4		
1/ Use only one CN source per line.		Totals =				

CN (weighted) = $\frac{\text{total product}}{\text{total area}}$ = _____ = _____; Use CN =

2. Runoff

Frequency yr
 Rainfall, P (24-hour) in
 Runoff, Q in
 (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)

Storm #1	Storm #2	Storm #3

TR - 20
JOB & TITLE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Watershed										Hydrologist										Date										SHEET										OF																																																																					
1-10										11-20										21-30										31-40										41-50										51-60										61-70										71-80																																							
1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0																																																												
JOB										TR-20																																																																																																			
TITLE																																																																																																													
TITLE																																																																																																													

JOB Record (required)

Columns 1-3 JOB

Columns 5-9 TR-20

Columns 11-17

HOLDOUT (not operational)
Use only if a holdout hydrograph is desired.

Columns 21-24 ECON

Use only if peak discharge output for ECON2/URB1 is desired. Locations for this output must be selected for inclusion in Summary Tables 1 and 3 also. Such designation can be made using summary option explained below (columns 51-57) or on Standard Control (column 71).

Columns 31-39 FULLPRINT

Use only if FULLPRINT option is desired.

Columns 41-48 PASS-XXX

Use only if PASS number, XXX, is greater than 1. XXX is an integer, right justified.

Columns 51-57 SUMMARY

Use only if Summary Tables 1 and 3 are to include all Standard Control operations (except SAVMOV).

Columns 61-67 NOPLOTS

Use if no cross section rating plots are desired.

ENDPLOT

Use if only cross section rating plots are desired with no routings.

Columns 73-80

Optional user information, printed at top of input listing.

TITLE Records (one required, second is optional)

Columns 1-5 TITLE

XXX (first TITLE re ord only).

An optional user file number, XXX, is an integer, right justified.

Columns 11-80

Both titles may be any characters desired.

TR-20

DISCHARGE VS. TIME

F-3

[illegible]

STANDARD SCS DIMENSIONLESS HYDROGRAPH

COMPUTED PEAK FACTOR = 484.00

4 DIMHYD	TIME INCREMENT			
	0.0200			
8	0.0	0.0300	0.1000	0.1900
8	0.4700	0.6600	0.8200	0.9300
8	1.0000	0.9900	0.9300	0.8600
8	0.6800	0.5600	0.4600	0.3900
8	0.2800	0.2410	0.2070	0.1740
8	0.1260	0.1070	0.0910	0.0770
8	0.0550	0.0470	0.0400	0.0340
8	0.0250	0.0210	0.0180	0.0150
8	0.0110	0.0090	0.0080	0.0070
8	0.0050	0.0040	0.0030	0.0020
8	0.0	0.0	0.0	0.0
9 ENDTBL				

COMPUTED PEAK RATE FACTOR = 484.00

The format for this table is Form #265, Page F-3.

TR - 20
STRUCTURE DATA
STRUCTURE NO. _____

**U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE**

[illegible]

TR - 20
STREAM CROSS SECTION DATA

**U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE**

CROSS SECTION NO. _____

[illegible]

TR - 20

CUMULATIVE RAINFALL TABLE

**U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE**

F-7

STORM TYPE OR DATE _____

Watershed _____										Hydrologist _____										Date _____										SHEET _____ OF _____									
1-10		11-20		21-30		31-40		41-50		51-60		61-70		71-80																									
1		2		3		4		5		6		7		8		9		0		1		2		3		4		5		6		7		8		9		0	
DATA		TABLE ID.		DATA FIELD NO. 1		DATA FIELD NO. 2		DATA FIELD NO. 3		DATA FIELD NO. 4		DATA FIELD NO. 5		RECORD IDENT.																									
CODE		NAME		NO																																			
5		R		A		I		N		F																													

RAINFALL TABLE 1

STANDARD SCS 24-HOUR, TYPE I DISTRIBUTION CUMULATIVE RAINFALL TABLE (REVISED MAY 1982)

TABLE NO.	TIME INCREMENT			
5 RAINFL 1	0.5000			
8	0.0	0.0170	0.0260	0.0350
8	0.0450	0.0650	0.0760	0.0870
8	0.0990	0.1260	0.1400	0.1560
8	0.1740	0.2190	0.2540	0.3030
8	0.5150	0.6240	0.6550	0.6820
8	0.7060	0.7480	0.7660	0.7830
8	0.7990	0.8300	0.8440	0.8570
8	0.8700	0.8930	0.9050	0.9160
8	0.9260	0.9460	0.9560	0.9650
8	0.9740	0.9920	1.0000	1.0000
9 ENDTBL				

Note: On Executive Control use Rainfall Depth in inches and Rainfall Duration of 1.0.
The format for this table is Form #271, Page F-7.

RAINFALL TABLE 2
STANDARD SCS 24-HOUR, TYPE II DISTRIBUTION
CUMULATIVE RAINFALL TABLE
(REVISED MAY 1982)

TABLE NO.		TIME INCREMENT			
5	RAINFLL 2	0.2500			
8		0.0	0.0020	0.0050	0.0080
8		0.0140	0.0170	0.0200	0.0230
8		0.0290	0.0320	0.0350	0.0380
8		0.0440	0.0480	0.0520	0.0560
8		0.0640	0.0680	0.0720	0.0760
8		0.0850	0.0900	0.0950	0.1000
8		0.1100	0.1150	0.1200	0.1260
8		0.1400	0.1470	0.1550	0.1630
8		0.1810	0.1910	0.2030	0.2180
8		0.2570	0.2830	0.3870	0.6630
8		0.7350	0.7580	0.7760	0.7910
8		0.8150	0.8250	0.8340	0.8420
8		0.8560	0.8630	0.8690	0.8750
8		0.8870	0.8930	0.8980	0.9030
8		0.9130	0.9180	0.9220	0.9260
8		0.9340	0.9380	0.9420	0.9460
8		0.9530	0.9560	0.9590	0.9620
8		0.9680	0.9710	0.9740	0.9770
8		0.9830	0.9860	0.9890	0.9920
8		0.9980	1.0000	1.0000	1.0000
9	ENDTBL				

Note: On Executive Control use Rainfall Depth in inches and Rainfall Duration of 1.0.
The format for this table is Form #271, Page F-7.

RAINFALL TABLE 3
STANDARD SCS 24-HOUR, TYPE IA DISTRIBUTION
CUMULATIVE RAINFALL TABLE
(REVISED MAY 1982)

TABLE NO.	TIME INCREMENT				
5 RAINFL 3	0.5000				
8	0.0	0.0100	0.0220	0.0360	0.0510
8	0.0670	0.0830	0.0990	0.1160	0.1350
8	0.1560	0.1790	0.2040	0.2330	0.2680
8	0.3100	0.4250	0.4800	0.5200	0.5500
8	0.5770	0.6010	0.6230	0.6440	0.6640
8	0.6830	0.7010	0.7190	0.7360	0.7530
8	0.7690	0.7850	0.8000	0.8150	0.8300
8	0.8440	0.8580	0.8710	0.8840	0.8960
8	0.9080	0.9200	0.9320	0.9440	0.9560
8	0.9670	0.9780	0.9890	1.0000	1.0000
9 ENDTBL					

Note: On Executive Control use Rainfall Depth in inches and Rainfall Duration of 1.0.
The format for this table is Form #271, Page F-7.

RAINFALL TABLE 4

 STANDARD SCS 48-HOUR, TYPE I DISTRIBUTION
 CUMULATIVE RAINFALL TABLE
 (REVISED MAY 1982)

TABLE NO.		TIME INCREMENT			
5	RAINFLL 4	0.5000			
8		0.0	0.0040	0.0080	0.0120
8		0.0200	0.0250	0.0300	0.0350
8		0.0450	0.0500	0.0550	0.0600
8		0.0700	0.0750	0.0810	0.0870
8		0.0990	0.1050	0.1110	0.1180
8		0.1320	0.1400	0.1480	0.1560
8		0.1740	0.1840	0.1950	0.2070
8		0.2360	0.2550	0.2770	0.3030
8		0.5150	0.5490	0.5830	0.6050
8		0.6400	0.6550	0.6690	0.6820
8		0.7050	0.7160	0.7270	0.7380
8		0.7580	0.7670	0.7760	0.7840
8		0.8000	0.8080	0.8160	0.8230
8		0.8370	0.8440	0.8510	0.8580
8		0.8700	0.8760	0.8820	0.8880
8		0.9000	0.9060	0.9110	0.9160
8		0.9260	0.9310	0.9360	0.9410
8		0.9510	0.9560	0.9610	0.9660
8		0.9760	0.9800	0.9840	0.9880
8		0.9960	1.0000	1.0000	1.0000
9	ENDTBL				

Note: On Executive Control use Rainfall Depth in inches and Rainfall Duration of 1.0.
 The format for this table is Form #271, Page F-7.

RAINFALL TABLE 5

STANDARD SCS 48-HOUR, TYPE II DISTRIBUTION
CUMULATIVE RAINFALL TABLE
(REVISED MAY 1982)

TABLE NO.		TIME INCREMENT			
5	RAINFALL	5	0.5000		
C	0.0	0.0020	0.0050	0.0080	0.0110
8	0.0140	0.0170	0.0200	0.0230	0.0260
8	0.0290	0.0320	0.0350	0.0380	0.0410
8	0.0440	0.0470	0.0510	0.0550	0.0590
8	0.0630	0.0670	0.0710	0.0750	0.0790
8	0.0840	0.0890	0.0940	0.0990	0.1040
8	0.1090	0.1140	0.1200	0.1260	0.1330
8	0.1400	0.1470	0.1540	0.1620	0.1710
8	0.1810	0.1920	0.2040	0.2170	0.2330
8	0.2520	0.2770	0.3180	0.6380	0.6980
8	0.7290	0.7520	0.7700	0.7850	0.7980
8	0.8090	0.8190	0.8290	0.8380	0.8460
8	0.8540	0.8610	0.8680	0.8740	0.8800
8	0.8860	0.8920	0.8970	0.9020	0.9070
8	0.9120	0.9170	0.9210	0.9250	0.9290
8	0.9330	0.9370	0.9410	0.9450	0.9490
8	0.9530	0.9570	0.9600	0.9630	0.9660
8	0.9690	0.9720	0.9750	0.9780	0.9810
8	0.9840	0.9870	0.9900	0.9930	0.9960
8	0.9980	1.0000	1.0000	1.0000	1.0000
9	ENDTBL				

Note: On Executive Control use Rainfall Depth in inches and Rainfall Duration of 1.0.
The format for this table is Form #271, Page F-7.

RAINFALL TABLE 6
STANDARD SCS DIMENSIONLESS DISTRIBUTION
for
EMERGENCY SPILLWAY ~~408~~ FREEBOARD HYDROGRAPHS
CUMULATIVE RAINFALL TABLE
(REVISED MAY 1982)

TABLE NO.	TIME INCREMENT			
5 RAINFL 6	0.0200			
8	0.0	0.0080	0.0162	0.0333
8	0.0425	0.0524	0.0630	0.0863
8	0.0990	0.1124	0.1265	0.1595
8	0.1800	0.2050	0.2550	0.4370
8	0.5300	0.6030	0.6330	0.6840
8	0.7050	0.7240	0.7420	0.7750
8	0.7900	0.8043	0.8180	0.8439
8	0.8561	0.8678	0.8790	0.9002
8	0.9103	0.9201	0.9297	0.9483
8	0.9574	0.9661	0.9747	0.9916
8	1.0000	1.0000	1.0000	1.0000
9 END18L				

Note: On Executive Control use Rainfall Depth in inches and Rainfall Duration in hours.
The format for this table is Form #271, Page F-7.

STANDARD CONTROL

Data Code/Operation

6 RUNOFF 1	Develops a runoff hydrograph using the data from Data Fields #1, 2, and 3.
6 RESVOR 2	Performs storage indication routing of a hydrograph. Data Field #1 is the starting elevation for routing to begin. If Data Field #1 is left blank, the routing will begin at the first entry in the STRUCT table.
6 REACH 3	Performs ATT-KIN reach routing using the reach length (Data Field #1) and, optionally, the end area coefficient, x , (Data Field #2) and the exponent, m , (Data Field #3) in the equation $Q = xA^m$. If Data Fields #2 and 3 are not used, a XSECTN table for the specific cross section is required.
6 ADDHYD 4	Adds two hydrographs together.
6 SAVMOV 5	Moves a hydrograph from one storage location to another storage location.
6 DIVERT 6	Divides an inflow hydrograph into two separate outflow hydrographs. Two procedures are available with this operation:
Procedure #1	Output #1 will contain that portion of the inflow hydrograph below and equal to the discharge given in the Data Field #1. Output #2 will contain that portion of the inflow hydrograph above the given discharge in Data Field #1.
Col. 25-36	Data Field No. 1 - Maximum discharge in cfs for Output #1 hydrograph.
Col. 37-48	Data Field No. 2 - Drainage area split in decimal fraction to retain with Output #1 hydrograph. If left blank, all drainage area goes with Output #2 hydrograph.
Col. 49-60	Data Field No. 3 - XSECTN ID associated with Output #2 hydrograph. (Note - ID must be provided in decimal form, i.e., 102.)
Procedure #2	The inflow hydrograph may be divided proportionally using two elevation vs. discharge rating tables. These tables must be provided by the user as cross sections.
Col. 13-15	ID of rating table associated with Output #1 hydrograph.
Col. 25-36	Blank or Zero.
Col. 37-48	Same as Procedure #1.
Col. 49-60	Same as Procedure #1 and a rating table must be given for the XSECTN.

Output Options

	If a particular option is wanted, put a "1" in the appropriate column.
Col. 61	PEAK - Will print Peak discharge, Peak Time and Peak Elev. (if rating table is available). Will supply up to the ten largest peaks of multi-peaked hydrographs.
Col. 63	HYD - Will print hydrograph discharges in tabular form.
Col. 65	ELEV - Will print hydrograph elevations in tabular form.
Col. 67	VOL - Will print the volume minus constant baseflow under the hydrograph in inches, cfs-hrs, and ac-ft and the current constant baseflow in cfs.
Col. 69	FILE - Will generate a file of the discharge hydrograph coordinates in machine readable READMD format.
Col. 71	SUM - Requests the results of the operation be inserted in Summary Tables 1 and 3 at the end of the job. ECON output must appear in Summary Table 1 but does not have to be selected here.
Col. 73-80	RECORD identification. Any characters including blanks.

STANDARD CONTROL FOR WATERSHED

[illegible]

NOTE: This form was suggested by members of Massachusetts Planning Staff

STANDARD CONTROL

Data Code/Operation

- 6 RUNOFF 1 Develops a runoff hydrograph using the data from Data Fields #1, 2, and 3.
- 6 RESVOR 2 Performs storage indication routing of a hydrograph. Data Field #1 is the starting elevation for routing to begin. If Data Field #1 is left blank, the routing will begin at the first entry in the STRUCT table.
- 6 REACH 3 Performs ATT-KIN reach routing using the reach length (Data Field #1) and, optionally, the end area coefficient, x , (Data Field #2) and the exponent, m , (Data Field #3) in the equation $Q = xA^m$. If Data Fields #2 and 3 are not used, a XSECTN table for the specific cross section is required.

6 ADDHYD 4 Adds two hydrographs together.

6 SAVMOV 5 Moves a hydrograph from one storage location to another storage location.

6 DIVERT 6 Divides an inflow hydrograph into two separate outflow hydrographs. Two procedures are available with this operation:

Procedure #1 Output #1 will contain that portion of the inflow hydrograph below and equal to the discharge given in the Data Field #1. Output #2 will contain that portion of the inflow hydrograph above the given discharge in Data Field #1.

Col. 25-36 Data Field No. 1 - Maximum discharge in cfs for Output #1 hydrograph.
Col. 37-48 Data Field No. 2 - Drainage area split in decimal fraction to retain with Output #1 hydrograph. If left blank, all drainage area goes with Output #2 hydrograph.
Col. 49-60 Data Field No. 3 - XSECTN ID associated with Output #2 hydrograph.
(Note - ID must be provided in decimal form, i.e., 102.)

Procedure #2 The inflow hydrograph may be divided proportionally using two elevation vs. discharge rating tables. These tables must be provided by the user as cross sections.

Col. 13-15 ID of rating table associated with Output #1 hydrograph.
Col. 25-36 Blank or Zero.
Col. 37-48 Same as Procedure #1.
Col. 49-60 Same as Procedure #1 and a rating table must be given for the XSECTN.

Output Options

If a particular option is wanted, put a "1" in the appropriate column.

- Col. 61 PEAK - Will print Peak discharge, Peak Time and Peak Elev. (if rating table is available). Will supply up to the ten largest peaks of multi-peaked hydrographs.
- Col. 63 HYD - Will print hydrograph discharges in tabular form.
- Col. 65 ELEV - Will print hydrograph elevations in tabular form.
- Col. 67 VOL - Will print the volume minus constant baseflow under the hydrograph in inches, cfs-hrs, and ac-ft and the current constant baseflow in cfs.
- Col. 69 FILE - Will generate a file of the discharge hydrograph coordinates in machine readable READMD format.
- Col. 71 SUM - Requests the results of the operation be inserted in Summary Tables 1 and 3 at the end of the job. ECON output must appear in Summary Table 1 but does not have to be selected here.
- Col. 73-80 RECORD identification. Any characters including blanks.

CONTINUATION SHEET

F-16

STANDARD CONTROL

Data Code/Operation

- 6 RUNOFF 1 Develops a runoff hydrograph using the data from Data Fields #1, 2, and 3.
- 6 RESVOR 2 Performs storage indication routing of a hydrograph. Data Field #1 is the starting elevation for routing to begin. If Data Field #1 is left blank, the routing will begin at the first entry in the STRUCT table.
- 6 REACH 3 Performs ATT-KIM reach routing using the reach length (Data Field #1) and, optionally, the end area coefficient, x , (Data Field #2) and the exponent, m , (Data Field #3) in the equation $Q = xA^m$. If Data Fields #2 and 3 are not used, a XSECTN table for the specific cross section is required.
- 6 ADDHYD 4 Adds two hydrographs together.
- 6 SAVMOV 5 Moves a hydrograph from one storage location to another storage location.
- 6 DIVERT 6 Divides an inflow hydrograph into two separate outflow hydrographs. Two procedures are available with this operation:

Procedure #1 Output #1 will contain that portion of the inflow hydrograph below and equal to the discharge given in the Data Field #1. Output #2 will contain that portion of the inflow hydrograph above the given discharge in Data Field #1.

- Col. 25-36 Data Field No. 1 - Maximum discharge in cfs for Output #1 hydrograph.
- Col. 37-48 Data Field No. 2 - Drainage area split in decimal fraction to retain with Output #1 hydrograph. If left blank, all drainage area goes with Output #2 hydrograph.
- Col. 49-60 Data Field No. 3 - XSECTN ID associated with Output #2 hydrograph. (Note - ID must be provided in decimal form, i.e., 102.)

Procedure #2 The inflow hydrograph may be divided proportionally using two elevation vs. discharge rating tables. These tables must be provided by the user as cross sections.

- Col. 13-15 ID of rating table associated with Output #1 hydrograph.
- Col. 25-36 Blank or Zero.
- Col. 37-48 Same as Procedure #1.
- Col. 49-60 Same as Procedure #1 and a rating table must be given for the XSECTN.

Output Options

If a particular option is wanted, put a "1" in the appropriate column.

- Col. 61 PEAK - Will print Peak discharge, Peak Time and Peak Elev. (if rating table is available). Will supply up to the ten largest peaks of multi-peaked hydrographs.
- Col. 63 HYD - Will print hydrograph discharges in tabular form.
- Col. 65 ELEV - Will print hydrograph elevations in tabular form.
- Col. 67 VOL - Will print the volume minus constant baseflow under the hydrograph in inches, cfs-hrs, and ac-ft and the current constant baseflow in cfs.
- Col. 69 FILE - Will generate a file of the discharge hydrograph coordinates in machine readable READHD format.
- Col. 71 SUM - Requests the results of the operation be inserted in Summary Tables 1 and 3 at the end of the job. ECON output must appear in Summary Table 1 but does not have to be selected here.
- Col. 73-80 RECORD identification. Any characters including blanks.

TR - 20
EXECUTIVE CONTROL FOR WATERSHED

**U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE**

Watershed _____ Hydrologist _____ Date _____																																																											
1-10			11-20			21-30			31-40			41-50			51-60			61-70			71-80																																						
1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0

DATA CODE	OPERATION	DATA FIELD #1	DATA FIELD #2	DATA FIELD #3	RECORD IDENT.
7	LIST	This record is used for printout of tabular data and standard control list that are currently effective			
7	BASFLOW	5	IF VOLUME IS IN DATA FIELD No 1 PEAK TIME, HOURS BASE TIME, HOURS		
7	INCREM	6	This increment is required for initial computations Remains in force until new increment is entered		

STARTING TIME HOURS		RAINFALL DEPTH INCHES	RAINFALL DURATION HOURS	ALTERNATE NO	EXAMPLES DO NOT ENTER
FROM	THRU				
OR STRUCTURE	OR STRUCTURE				
7	COMPUT	7			
	ENDCMP	1	This record is required at "end" of a watershed, if may follow a series of COMPUT records		
	ENDJOB	2	This record terminates routing computation and precedes the PEAKS record, if any		

Right justify these fields:

IMPORTANT: Data Fields No. 1, 2, and 3 require decimal points.

EXECUTIVE CONTROL

Data Code/Operation

7 LIST	Tabular data and Standard Control (S. C.) instructions in effect at the time are printed.
7 BASFLO 5	Baseflow is added to an inflow hydrograph.
Col. 25-36 or	Data Field #1 - Insert the baseflow CFS to be part of the inflow hydrograph in the next and subsequent reach routings until changed by another 7 BASFLO 5 record.
Col. 25-36	Data Field #1 - The volume in inches in triangular hydrograph form that is added to each runoff hydrograph computed. For this procedure, Data Field #2 columns 37-48 and Data Field #3, columns 49-60 must be used.
Col. 37-48	Data Field #2 - Time in hours that the triangular hydrograph peaks. (Note: If a field is left blank or zero, constant baseflow procedure is used).
Col. 49-60	Data Field #3 - Time in hours that the triangular hydrograph base extends. (Note: Cannot exceed 298 main time increments.)
7 INCREM 6	Sets the time interval between elements of the hydrograph to be used for subsequent computations. This record must be read before any COMPUT record.
Col. 25-36	Data Field #1 is time increment in hours. A decimal is required.
7 COMPUT 7	Specifies the beginning and ending S.C. instructions to be executed. One pass through the watershed (S.C. list) requires one or more COMPUT Records.
Col. 13-17	Specifies the beginning location: This must correspond to a cross-section or structure number that appears in the XSECTN/STRUC column of an S.C. instruction.
Col. 19-22	Specifies the ending location. The sequence or instruction given by the S.C. is continued until the cross-section or structure specified under Thru has been reached and completed.
Col. 25-36	Data Field #1 is the beginning time in hours of each hydrograph developed under this COMPUT. A decimal is required.
Col. 37-40	Data Field #2 is the rain depth to be used. If a unit depth (dimensionless) RAINFL table is used, enter the total rain depth in inches. If an actual storm is used, enter a 1.0. A decimal is required.
Col. 49-60	Data Field #3 is the duration of the rain table to be used. If a unit time (dimensionless) RAINFL table is used, enter the total duration in hrs. If a time increment in hours is used in the RAINFL table, enter a 1.0. A decimal is required.
Data Field #4 contains additional data, as follows:	
Col. 61	The RAINFL table to be used (1-9).
Col. 63	The Antecedent Moisture Condition to be used (1, 2, or 3).
Col. 66-67	The alternate number (1-99) associated with this COMPUT. Usually reflects changes in the watershed.
Col. 70-71	The storm number (1-10) to be associated with this COMPUT.
Note: The last two data items are used to order the summary table output data and for generating the peak discharge file in machine readable form for use in the ECON2 or URBI computer programs.	
ENDCMP 1	Resets the program to the beginning of the S.C. list. This record must be used anytime it is desired to start another pass through the watershed.
ENDJOB 2	Terminates routing computations. Causes Summary Table 2 to be printed and Summary Tables #1 and #3 if selected. Job is terminated unless PEAKS information follows.